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HIGHWAY ENGINEERING

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Introduction

Modern highways are complex engineering structures; the calculations providing the basis for the design of individual road elements are often just as complicated as the design calculations for machine components, bridges and the structural details of public and industrial buildings.

Modern highways are intended for high-speed motor traffic. Therefore, they must be designed and constructed in such a way that the performance characteristics of vehicles may be effectively realized under normal conditions of engine operation. Their design should permit vehicles to negotiate bends and gradients without the danger of skidding and overturning and without causing fatigue and discomfort to passengers. The road pavement must continuously provide good riding qualities and be capable of withstanding the dynamic loads induced by the passage of vehicles.

Pavements and road subgrades are subject to the influence of many natural factors, e.g., heating by the sun, freezing and thawing, moistening by rain, etc. In the annual cycle, complex physical processes develop in the subgrade occasioned by the variation in moisture distribution and an increase in subsoil moisture content. An excessive moisture content quickly causes the subsoil to lose its strength and may lead to disintegration of the road foundation. The many and varied factors of pavement performance have to be taken into account by the designer and constructor, who have to provide for the maximum stability of the subgrade and for the maximum strength of the pavement to be laid thereon.

Roads are built in the most varied natural conditions—in the broad plains and hills of the European part of the U.S.S.R., amidst the lakes, marshes and rocks of Karelia, in the regions of Siberia covered by taiga forests, on permafrost subsoils, in the sandy deserts and irrigated cotton plantations of Central Asia, in the mountains of the Caucasus and Pamir, in the fertile virgin lands of Siberia and Kazakhstan, and in the black earth steppes of the Ukraine and Kuban. A similar diversity of terrain exists in all other countries having a large territory.

In all these diverse and complex conditions the road engineer has to be able to find the correct engineering and economic solutions. Because of this, when solving problems related to road construction,

he has to make use of natural and historical sciences, i.e., geology, climatology, soil physics and mechanics, surveying, hydraulics, hydrology, etc.

At present the requirements for an engineer engaged in the design and construction of highways are very exacting. He must be fully conversant with the methods of route selection and with the methods used to obtain field data required for design purposes. He must be able to design highways so as to ensure the comfort and safety of transportation. At the same time he must take into account to a maximum extent the local geophysical conditions which influence the construction and maintenance of highways.

The maximum use should be made of modern machinery in the best possible combination for roadbed and pavement construction.

Finally, when the highway is put into service, its maintenance and the provision for uninterrupted traffic become of the utmost importance for the national economy. The engineer in charge of the highway operation must ensure the maintenance of the road quality under all traffic and weather conditions. He must be familiar with the methods of counteracting the natural agents which threaten the road stability and which can interrupt the traffic (snow and sand drifts, frost heave, washouts by rain, landslides, floods, etc.).

Road jobs are essentially labour-consuming, demanding the extensive transportation of large quantities of materials. Thus, for the construction of 1 km of a motor road with asphalt-macadam surfacing on a gravel base, in flat country, it is necessary to transport about 7,500 tons of sand and gravel and excavate up to 12,000 cum of soil, transporting it for a distance of perhaps several hundred metres. Stone aggregates used in the road pavement often have to be hauled from far afield.

The road-building operations become complicated because of the extensive length of the construction site—often tens and hundreds of kilometres. This requires the introduction of special techniques and methods of work organization.

The task of the road engineer is to mechanize and technically develop the road-building operations, and to provide for the most efficient and complete mechanization of the entire construction process.

As in other fields of construction, road building requires the application of industrialization techniques on a wide scale—the use of prefabricated reinforced concrete structures, light-weight concrete constructions, factory-made large blocks and assemblies. Because of this, road construction and the building of artificial structures form complementary parts of the same constructional programme.

The mechanization of road construction has grown immensely since the war. This is true of such operations as earthworks, sand and gravel quarrying, stone crushing and the completion of asphalt macadam and cement concrete surfacing.

The diversity of natural conditions in many countries—sharp variation of climatic, ground, and hydrological conditions of various regions—precludes the use of standardized design methods. Designers must have a creative approach to their problems; they must thoroughly assess the influence on the constructed road of natural agents and of the loads imposed by vehicular traffic.

The solutions of road construction problems are closely allied to those of reduction of cost and the improvement of the quality of the work. To ensure the most economical design of the road it is necessary to assimilate the experience gained in the carrying out of similar projects. It is very important that the latest techniques developed in the fields of science and engineering be applied to the construction and analysis of road and bridge projects.

When designing a highway one should reject over-large safety factors, and limit the consumption of allocated and imported materials. Extensive use should be made of local materials of limited strength, including local soils, by employing them in structures where the stresses caused by traffic and natural agents will permit their use. One must envisage the employment of chemical and physical stabilization of soils when necessary in order to increase their strength and stability.

It is imperative to extend the scientific basis of road building. The construction of highways in complex natural conditions constantly demands the continued scientific approach to new problems.

At present, the principles of road design and construction in difficult conditions are not fully developed in all their details. Great difficulties may be encountered in the use of local materials of limited strength, which may result in frequent failures.

The science of road construction is on the threshold of new developments. Since it is an applied science, it depends on the achievements of physical, mathematical and natural sciences. A wider application of these sciences linked with the future important development of the chemical industry opens up great possibilities for road engineers to be able to alter the properties of local soils and stone aggregates by means of physical and chemical treatment of their active colloidal constituent parts.

The road engineer should be prepared for the possible alteration, in the near future, of the character of vehicular traffic on roads. Development in electric supply could permit the introduction of trolleybus services on country roads, and the use of radar may give rise to automatic traffic control. One possibility of a widespread

development in the use of atomic energy would be that the road engineer may have the opportunity of obtaining monolithic surfacing by the fusion of local soils.

With the growth of traffic density will come a demand for greater amenities. It will be necessary to provide the national road system with hotels and restaurants, service stations and repair shops, where drivers and passengers may rest, and the vehicles be serviced and overhauled. Attention should be given to such questions as fitting roads into the landscape in plan and in profile, planting trees, so as to improve their aesthetic value.

Brief Survey of Road Engineering Development

Road engineering is one of the earliest arts known to mankind. The development of industry and the improvement of the means of transport have led to the alteration and improvement of road building and of methods of road construction.

Roadmaking originated in the period of early human settlements. People would choose the most convenient and the shortest ways of approach to their hunting and fishing grounds, gradually making footpaths. The earliest bridges were naturally fallen trees across waterways; gradually, however, crossings were built of logs. With the use of tamed animals for transport, the paths had to satisfy higher standards since bridle paths for pack animals must be cleared to a greater width and height.

The first artificially constructed tracks were made in mountainous and forested country, where obstacles to movement were encountered. It is likely that the first road surfacing was simply a layer of logs and brushwood over marshland.

About four to five thousand years B. C. the introduction of the wheel constituted a major technical achievement and greatly accelerated the development of road construction. The possibility of carrying a greater load on wheels than could be moved by dragging it, called for a corresponding improvement in carriageway and bridge flooring and also created a demand for more convenient road alignments and the bypassing of marshland and loose sands.

Road construction received a substantial impetus during the slave-owning period of the ancient world—in Assyria, Babylon, Persia and especially in the Roman Empire. This road building was maintained because of the continuous warfare with neighbouring states, which required roads to link the centre of the country with its borders. Thus, the transition from footpaths and bridle tracks to comparatively well built roads was achieved largely through m litary considerations.

Commercial traffic at that time went mainly by sea and river. These routes were cheaper than transport by land and were safer. This situation was further accentuated by the territorial dissociation of the various states and the absence of an interconnected system of land communications. Therefore, during the whole of the slave-owning period, and later of the feudal epoch, water transport developed more quickly than overland transport. However, an appreciably well developed road network was laid out in places where there were no waterways.

The territory of the contemporary Uzbek and Turkmenian republics was crossed by great caravan routes which served for trade between the people of Central Asia and China, Greece and Rome. These routes consisted of wide tracts of free land, within the limits of which there was grazing fodder for draught and pack animals. The way was marked only by wells and inns, with fords and isolated bridges at crossings of waterways.

The construction of the stone arch bridge originated during the ancient Asiatic epoch. The earliest bridges had pointed arches, e.g., the ancient bridges of Persia, but later the Romans developed the semi-circular arch which they used on bridges and viaducts.

In the ancient civilizations of the East artificially paved surfacings were used mainly for town streets and approaches to temples. Baked bricks were used extensively for paving in Assyria and Babylon, as well as mastic asphalt—a mixture of natural bitumen, clay, sand and gravel. Limestone slabs were also used for street pavement.

Road construction was extensively developed during the Roman Empire, the strategic and commercial aims of which required the creation of lines of communication cutting across Europe (Fig. 1). The Roman historian Tacitus says that the roads of that time were required by "the traders and the Roman army", and the roads they constructed were proof of the might of the Roman Empire. The Roman highway network, built during seven centuries, extended over a total length of 90,000 km, of which 14,000 km were situated within present-day Italy. If one is to take into account secondary earth and gravel roads, the total length of the Roman Empire road network would attain 300 thousand kilometres.

The major Roman roads were of solid stone construction incorporating up to 10 to 15 thousand cubic metres of stone per kilometre. This is from 4 to 6 times the amount used today in the construction of modern motorways (Fig. 2).

Materials used in the construction of Roman roads were gravel, cobblestone and hewn stone in the form of slabs. Lime burning was known to the Romans, who used concrete extensively for constructional purposes, employing as the matrix a mixture of lime, loose volcanic rock (pozzuolana) and sand.

The Romans were skilled in the art of bridge building as evidenced by their roads which were endowed with innumerable stone arch bridges whose remains can still be found in Italy, France and Spain.

As a rule, Roman roads were aligned to provide the most direct route ignoring natural obstacles. This policy necessitated the construction of numerous structures. For example, a depression 35 m deep was filled in along the Appian Way near Terracina, whilst near Naples the Romans drove a tunnel 1,300 m long, 10 m high,

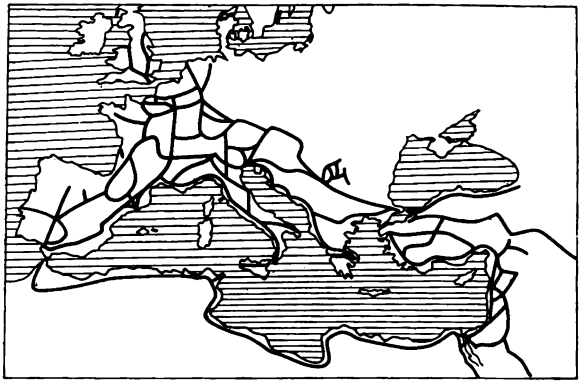


Fig. 1. European major road network—2nd century A.D.

and 8 m wide. At intervals of 10 to 15 km along these roads, inns were sited where about 40 horses were kept; by changing horses messengers were able to cover up to 150 km per day.

The ruthless exploitation by the slave-owning states of conquered provinces caused a gradual decline in their forces of production. As F. Engels noted, this led to "universal impoverishment; decline of commerce, handicrafts, the arts and of the population; decay of towns; retrogression of agriculture to a lower stage". The Roman Empire, weakened by the slave revolts, was conquered at the end of the fifth century by the Germanic barbarians, and in its place appeared dozens of small feudal states. Within the limits of the separate states and domains the economies were of the subsistence type. The European arterial roads, which now crossed several states, lost the importance they once held during the Roman era, and were allowed to fall into decay.

The importance of land communications grew appreciably at the end of the feudal period, when the process of uniting various separate feudal domains into great states took place.

In the second half of the 18th century a period of intensive road building began, the rate of building being dependent upon the rate of development of industry and commerce in various states. The construction of roads with uniform hard surface substantially

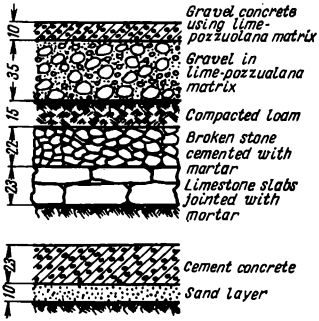


Fig. 2. Comparison of pavement structures of Roman roads and contemporary highways. Above is the pavement of a Praetorian road; below is a modern cement concrete pavement for a traffic flow of 5,000 vehicles per day

improved the conditions for the transportation of raw materials and of finished products by reducing the tractive resistance and hence allowing an increase in the load carried by individual vehicles.

At first, roads similar to the Roman roads were built. However, owing to a scarcity of suitable material and the high cost of labour, the amount of stone material used was progressively reduced and the work was carried out less thoroughly. Research was undertaken with a view to finding more rational methods of using stone for pavement construction which would reduce both the amount of labour and the cost.

Important stages in the development of road pavement construction were marked by the introduction of two types of construction, called by the names of their inventors—the Frenchman Tresaquet and the Scot McAdam. Tresaquet's system consisted in building the road pavement in a wide trench dug out of the natural ground. The

bottom of the trench was given a camber in order to divert the water seeping from above. The pavement base was of uniform thickness for the whole width of the carriageway and consisted of slabs placed on end (Fig. 3*a*). The surface course was of crushed aggregate of the hardest rock. The stone pavement thickness was now reduced to 0.24-0.27 m instead of 1 m as it was customary in Roman road construction.

McAdam proposed to build roads of a granular base 25 cm thick, which were to be compacted by the rolling carriages (Fig. 3*b*). The

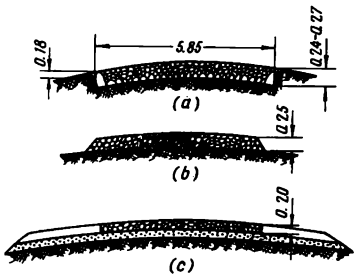


Fig. 3. Construction of stone aggregate pavements at the end of the 18th and at the beginning of the 19th century:

a—Tresaquet's pavement (France), end of the 18th century; *b*—pavement designed by McAdam (Great Britain), 1830; *c*—granular pavement on a sand base

granular base was laid on a thoroughly levelled and compacted formation which ensured the elimination of water. McAdam was the first to observe that the strength of the pavement could be assured only when the subgrade resistance to loading was reliable.

Russian engineers were the first ones (1836) to construct granular surfacings laid on a sand base, which are now in fairly wide use in other countries (Fig. 3*c*). The use of a sand base permitted not only a reduction in the cost of construction, since much of the costly stone material was not required now, but also facilitated the removal of water from the subgrade. The latter increases subgrade stability and prevents the formation of frost heaves which are a specific phenomenon of pavements in spring, and are the result of excessive moisture content in the subgrade.

The construction of hard-surfaced roads led to the mechanization of transport, because horse-drawn transport could no longer cope with the increased goods traffic. In a number of countries attempts

were made to introduce steam traction engines. However, because of the heavy weight of the steam engines, their imperfect construction and the lack of adaptability of roads for mechanical transportation, these efforts to introduce steam-driven vehicles onto highways were not successful at the beginning of the 19th century. The demand for transportation was satisfied by the construction of railways on a grandiose scale, thus relegating the highways for horse transport to the secondary role of approach roads to the railway stations.

The type of road designed in the middle of the last century, with its granular surfacing, completely satisfied the requirements of horse transport. The engineering science and technique of granular surfacing had improved appreciably, the practice of laying surfacings with a uniform grain size compacted by vehicles being gradually superseded by surfacing in which the granular material was blinded during rolling with loose fine stone aggregate—siftings and screenings. Mechanical stone crushing and steam rolling also came into use.

With the advent of the mechanically-propelled road vehicle at the beginning of the present century, it was necessary to alter radically the construction of road pavements. Granular surfacings, the strength of which was ensured by the wedging effect accomplished by rolling, began to deteriorate rapidly under the impact of automobile traffic. Therefore, the construction of more stable roads was necessary, and use was made of stone materials bound by bitumen or coal tar.

Attempts to use organic binding materials for road pavements were made in the first half of the 19th century. However, at that time their application was aimed primarily at reducing dust on the roads and the noise of the traffic. With the coming of automobile traffic the road surfacing had to satisfy a new requirement—the resistance to tangential forces developed upon the transmission of the torque by the wheels of the vehicles. This could be satisfied only by the introduction of special materials which cemented the aggregate together.

As long as the volume of automobile traffic was not great, roads could serve simultaneously for both mechanically-propelled and horse-drawn transport. The requirements of motor vehicles were taken into account by partially adapting the alignment and profile of the road, viz., increasing the radii of horizontal curves and eliminating sharp breaks in profile. Roads of that period were sometimes called "autocarting" roads.

The period after the world war of 1914-1918 was marked by the quick growth of automobile transport and an increase in the speed and carrying capacity of the automobiles. It became obvious that

intensive automobile traffic and horse traffic could not be combined on mixed-purpose roads. Therefore, parallel with the construction of mixed-purpose roads on secondary routes, highways were constructed which were intended exclusively for high-speed automobile traffic on a large scale, i.e., motorways and expressways, all the elements of which were designed for high-speed traffic.

Expressways are roads intended for the transportation of passengers and goods over extended distances by motor vehicles, and which permit such journeys to be accomplished without obstruction from local traffic.

The expressways are provided with motels and service stations. On the modern expressway the high speed of traffic makes it imperative that the two opposing streams of vehicles should be physically separated. Therefore, expressways are built with dual carriageways, each of which should have a minimum width of two traffic lanes.

On an expressway there are no level crossings, no traffic lights and no signals requiring the vehicles to stop. The entry to expressways is possible only by special approach roads.

The economic committees of UNO have developed projects of European and Asian International Highway Systems which include the main expressways of individual countries.

PART I

The Road. General

CHAPTER 1

THE HIGHWAY NETWORK

1. Highways and the National Transport System

The transportation of passengers and goods is accomplished, in practice, by means of a *communication network* consisting of railways, highways, aircraft routes, and river and sea routes. In countries with a planned economy all means of transport form a single transportation system and their operations are coordinated, thus complementing each other's services and providing an opportunity to rationalize the use of each service.

The main volume of long-distance commercial and passenger traffic is carried by rail transport. However, goods handled by rail are received and delivered at special freight stations. Therefore, railways have to operate in conjunction with other forms of transport, which function on the approach roads to the railway lines. Approach roads are also required to service sea, river and canal transport and airports, the role of approach roads being played by motor roads and highways.

Goods may be loaded on to motor vehicles directly at the place of their production, and these goods may then be carried without transfer directly to their respective destinations. Because of this, motor transport is the most efficient form of transportation over comparatively short distances. Depending on the nature of the road network, the delivery of freight for a distance of 200 to 400 km is accomplished more quickly by road than by rail.

The total volume of goods carried by motor transport is appreciably greater than that transported by all other means of transport.

Motor transport plays an important part in the development of sparsely populated districts, providing for the transport of goods while at the same time keeping the costs of road construction com-

paratively low. In recent years, with the construction of modern highway networks, motorized transport has also acquired importance as a means for the long-distance transportation of passengers and freight.

2. Highway Network Fundamentals

Roads which interconnect inhabited localities and industrial and agricultural centres, linking them to freight handling stations for other means of transport, constitute the basic *highway network.* Persons and goods requiring to be transported between specific origins and destinations, the amount of goods depending on the requirements of the national economy and established trade relations, make up traffic streams.

In planning an effective automobile highway network it is essential, in the first instance, to take into account the main freight and passenger traffic streams in order to keep the costs down and to facilitate the delivery of goods. The framework of a highway network is a system of trunk roads designed for long-distance high-speed passenger and goods traffic, and connecting the main economic regions of the country with its basic economic centres.

When laying out a highway network it is essential to maintain administrative, cultural and economic communications between various parts of the country.

The location of a highway network is a fundamental element of road planning, and is determined by the distribution of the country's productive forces, the further development of which it must promote. However, the considerable amount of money already invested in road building compels the designer to make maximum use of the existing metalled roads. In all projects concerned with the development of highway networks, therefore, considerable attention must be given to the reconstruction of roads in order to render them suitable for modern high-speed motor traffic.

3. Characteristics of Highway Traffic

Vehicles travelling in the same direction constitute a *traffic stream.* It is apparent that the greater the number of vehicles in a stream, the more severe will be the requirements to be satisfied by the road.

A traffic stream usually consists of many types of vehicles, travelling at different speeds and carrying various loads. However, in order to determine the layout of the road, e.g., the width of the carriageways and the overall width of the road, the total number of vehicles on the road at a given period is taken as the major design criterion, and not the loads they may be carrying. The total number

of vehicles passing through any section of a road in unit time (day, hour) is called the *traffic intensity* or *flow* and is the measure of traffic usage for design purposes. Traffic flow varies along each individual road section; it increases in the vicinity of towns, large inhabited localities and railway stations, and is reduced along stretches of road some considerable distance from large towns and cities (Fig. 4*a*). The traffic flow on a road does not remain uniform throughout the year. When seasonal agricultural activity is high, especially during the harvest, traffic intensity on country roads builds up appreciably (Fig. 4*b*). On the other hand, the volume of freight traffic decreases during holiday periods. Nor does traffic flow remain constant throughout the day, and it decreases sharply at nightfall (Fig. 4*c*). As a result of the practical measurement of traffic, it is customary to accept for calculation purposes that the total daily volume of traffic passes during ten hours of daytime.

In the project stage of highway design, traffic is usually described in terms of the "annual mean daily flow" (A.D.F.), i.e., the total number of vehicles passing per year divided by 365. In some cases, e.g., when planning a road for the transportation of agricultural produce (grain, sugar beet, etc.), traffic flow at harvest time may be appreciably in excess of the mean daily flow. In view of the national importance of this traffic, it would be desirable in this instance to base the layout and geometric design of the road on flow during the peak period.

The total traffic intensity in both directions during the peak hour may also be taken for purposes of design.

Traffic flow is not the only basic traffic characteristic. One has to bear in mind other factors when resolving certain design and operational problems of highways.

To determine the pavement thickness and to design different structures one has to know not only the number of loads, but also their weight. This makes it necessary to divide the total traffic flow into separate streams according to the load-carrying capacity of the vehicles.

For design purposes, motor vehicles in the U.S.S.R. are divided into four basic categories:

Very small capacity —up to 1.0 ton
Small capacity —from 1 to 2 tons
Medium capacity—from 2 to 5 tons
Large capacity —from 5 tons to the limit permitted by road vehicle regulations

Soviet trucks of very large capacity—MAZ-525 and MAZ-530 of 25 and 40 ton capacity—are intended for use at quarries and construction sites and can be seen on general purpose roads only on rare occasions.

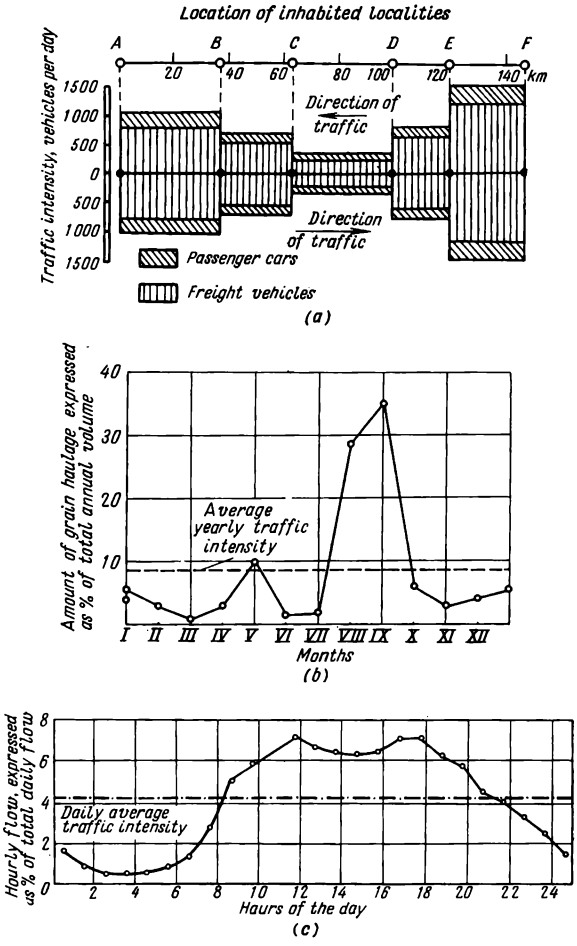


Fig. 4. Variations of traffic flow on highways:
a—on various sections; b—during a year; c—during a day

The maximum wheel loads and overall dimensions of single and combination vehicles are established in the U.S.S.R by State Standards.

Single and combination vehicles are divided into two groups. Group A includes vehicles operating on roads of classes I and II with improved pavements, as well as on roads of other classes if they are specially designed for such vehicles. Group B includes all other single and combination vehicles.

All vehicles whose weight when loaded exceeds 1.5 tons must have wheels with pneumatic tyres, with a mean unit load on the pavement of not over 6.5 kg/cm² for group A and 5.5 kg/cm² for group B.

The principal parameters of vehicles are given in the following table:

Parameter	Group A	Group B
Maximum single-axle load, t:		
(a) With distance between axles of 3 m and above	10	6
(b) Ditto for buses with full load	11.5	7
(c) Ditto for dump trucks	—	6.5
(d) Ditto with distance between adjacent axles less than 3 m	9	5
Total weight of vehicle, t:		
(a) Two-axle vehicle or trailer	17.5	10.5
(b) Three-axle vehicle or trailer	25	15
(c) Three-axle combination vehicle (tractor with semi-trailer)	25	16
(d) Ditto, four axles	33	20
(e) Ditto, five axles	40	30
Total length, m:		
(a) Vehicle with any number of axles (without trailer)	12	12
(b) Combination vehicles with one trailer	20	20
(c) Ditto with two or more trailers	24	24

Theory and practice show that a heavy-weight vehicle at one pass can do more damage to a road than the passage of a great number of lighter vehicles. These circumstances are allowed for in the design of pavements by the actual traffic intensity being expressed in *equivalent units* of one of the vehicles taken as a standard (see Sec. 52).

The amount of attrition of the road surface is dependent upon the total weight of vehicles which have passed since the road was last repaired. Because of this, traffic flow is measured in terms of the *gross laden weight* of vehicles traversing the road.

4. National and Functional Classification of Highways

The importance of motor roads for the national economy is, in the majority of cases, closely related to the intensity of traffic on them, i.e., the higher the flow the better should be the standard of design. Where flows are heavy, the expenditure necessary for the construction of the road to follow the most direct route and with shallow gradients will soon be compensated by the economy in traffic operation. On the other hand, if in spite of a high traffic flow the road is built with steep gradients and a narrow carriageway, though its capital cost may be much lower it will not permit the most effective performance of vehicles to be realized, in particular the maintenance of high vehicle speeds. In the long run, the cost of motor transport operation would become excessive.

The question of choice of the type of road, however, does not depend exclusively on the cost of construction. A number of other major factors must be taken into consideration, particularly the part played by the specific highway in the transport system of the national economy. Therefore, it becomes necessary to have two road classifications: a *national* one in accordance with the specific importance of the road for the national economy, with a view to both present needs and future development, and a *functional* one based on the traffic flow, which may not necessarily be coincident with the national one.

An example of a national classification of motor roads is the one applied in the Soviet Union, where the various roads are divided into the following groups according to their importance for the national economy and cultural life of the country, as well as according to administrative needs:

1. Arterial roads of all-Union importance, intended for long-distance motor communications between large centres and remote economic regions.
2. Arterial roads of republican importance for long-distance motor communications between remote regions of the Union republics.
3. Highways of regional importance, serving to connect districts and large enterprises with regional centres, railway stations and docks.
4. Roads serving district needs, connecting district centres with other inhabited centres and large local industrial enterprises, with

state farms, collective farms, railway stations and docks. The building and maintenance of these roads are carried out by the local organizations.

In the U.S.S.R. the regional and district roads carry the bulk of haulage, since their cumulative length forms 80% of the total road network. In certain cases these are roads with a pavement of inferior quality.

The reconstruction and building anew of the most important parts of this road network, i.e., roads adjoining large towns and industrial centres, approaches to railway stations and docks, should receive priority in road construction plans for the near future.

5. Resort roads, mainly for passenger traffic within health resort districts.

6. Approach roads to large towns and industrial centres, linking them with neighbouring districts.

7. Town roads and roads in inhabited places (streets) which serve the internal passenger and goods traffic. These roads are the responsibility of the municipal services.

8. Roads used by separate economies and enterprises, and approach roads carrying internal traffic.

There is a number of roads which are built according to high technical standards in spite of their comparatively low traffic intensity, e.g., roads within health resort areas which offer high-standard amenities to holiday-makers and patients. These roads are always of the highest technical standard.

The expected traffic intensity cannot be the only criterion when designing roads for construction in new, sparsely populated regions. In spite of the expected low traffic intensity for a number of years to come, such roads will constitute the main artery for populating these regions. Therefore, pioneer roads can be built with a view to district development, according to technical standards corresponding to a traffic intensity exceeding the present rate.

The motor roads of the U.S.S.R. are divided into five technical classes. The class is determined according to the importance of the road for the national economy. At the same time potential traffic intensities are considered, as well as the construction difficulties arising from the topographic features of the country in which the road is to be located.

The elements of the plan, profile and cross-section are designed with a view to the traffic intensities expected in 20 years, and the road pavement—in 5 to 10 years, depending on its construction and the possibility of gradually strengthening it.

Class I comprises roads having special economic, administrative or cultural importance for the national economy of the U.S.S.R. and having a high initial or potential traffic intensity; class II

comprises similar roads with an appreciable potential traffic flow; class III covers motor roads with a moderate traffic flow but having a very great importance for the national economy of the Union republics; class IV includes roads having local economic, administrative or cultural importance and a low traffic flow, and class V covers motor roads with small initial and potential traffic flow.

In the particularly difficult conditions of a mountainous region it is permitted, provided one can justify this on economic grounds, to lower the classification of a road at especially difficult sections by one class. Table 1 gives the road classes in relation to potential traffic flows.

TABLE 1

Highway Classification System in the U.S.S.R.

Potential intensity of vehicular traffic (annual mean daily flow=A.D.F.)	Technical class of road
More than 6,000 vehicles	I
From 3,000 to 6,000 vehicles	II
From 1,000 to 3,000 vehicles	III
From 200 to 1,000 vehicles	IV
Less than 200 vehicles	V

All road elements of each technical class are designed to ensure the safe running of individual passenger cars under normal conditions of cohesion between vehicle wheels and the carriageway surface (a dry or comparatively clean wet pavement surface).

The geometric design of class I roads is based upon a design speed of 150 km/hr. Class II roads have a design speed of 120 km/hr, class III roads—100 km/hr, class IV roads—80 km/hr and class V roads—60 km/hr.

On difficult sections of rugged country the design speed is reduced by 20 km/hr, while on difficult sections of mountainous terrain it is halved (80 km/hr for roads of class I).

The design traffic speed for class I roads corresponds to the actual speeds of modern motor cars, e.g., ZIL-110 and GAZ-12, and is lower than the probable speeds of passenger cars to be produced in the near future. Therefore, when laying out motorways intended mainly for high-speed passenger traffic, the design speed may be increased to 160-180 km/hr. When designing roads an attempt should be made to allow a traffic speed exceeding the rated one, except

when this entails substantial increases in constructional cost. This is especially important in the case of class III or IV roads.

In general the design speeds accepted in the U.S.S.R. correspond to those used in other countries. For example, on expressways in Western Germany the design speed in relation to topographic features is 160 km/hr in flat country, 140 km/hr in hilly country and 120 km/hr in mountainous areas. In Great Britain the design speed for motorways is taken as 130 km/hr, in the U.S.A. it is 112 km/hr. The UNO Economic Commission for Europe recommended for the International Highway System a design speed of 120 km/hr.

CHAPTER 2

HIGHWAY DESIGN

5. The Road in Plan

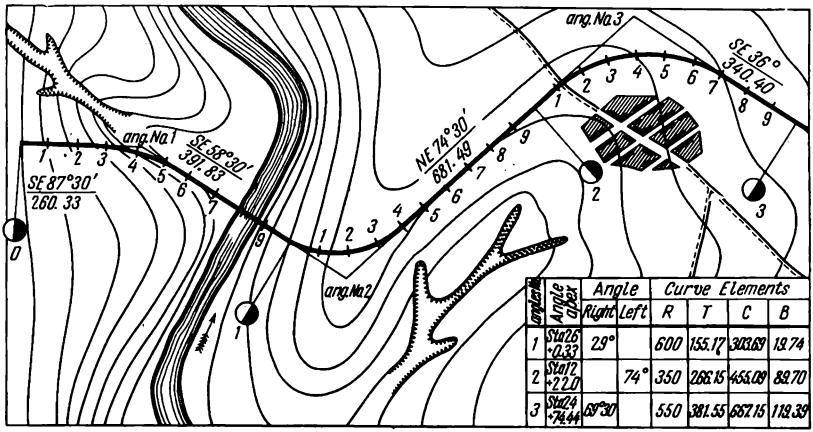
Highways are designed for the haulage of goods and passengers with a minimum of effort and at low cost. These requirements would be satisfied best if the road could be built along the shortest distance, i.e., a straight line between two given points. However, the building of a road along the shortest distance is precluded by the topography of the land (mountains, ravines, etc.), water obstacles (marshes, lakes, rivers), as well as the necessity to lay the road through certain intermediary points—places adjoining towns, places conveniently located for crossing rivers, railways or other highways.

As can be seen from Fig. 5, the necessity to locate the crossing where the river is straight and affords a convenient approach with shallow banks, the desirability of bypassing an inhabited locality and the necessity to avoid the crossing of a ravine dictated the location of the road along the broken line of the plan rather than along the shortest and most direct (air line) route. For the convenience of passage of motor vehicles, it is necessary to inscribe circular arcs of adequate radius at changes in direction.

Such a line, marked on the land and located along the road centre line, is called the *route*. The graphical representation of the line of the route, projected on a horizontal plane and drawn to a given scale, is called the *plan of the route*.

Any deviation of the direction of the route is determined by the *deflection angle*, which is measured between the continued previous line of the route and its new direction (Fig. 6). In practice, deflection angles are given consecutive identifying labels. In order to transfer the projected line of the route on to the ground, the bearings of the individual straight sections of the route are carefully determined in relation to the cardinal points. This facilitates the production of a route plan which may be accurately oriented.

Conditions for the high-speed driving of vehicles tend to deteriorate on curved sections, especially on bends of small radius, since steering becomes more involved. When moving along a curve, the motor vehicle is subject to a centrifugal force, the effect of which tends to displace it off the road, and to prejudice the car's stability. Also, the driver's road visibility is impaired; in some cases, the plantings at the side of the road have to be cleared, or the faces of



angle No.	Angle apex	Angle Right	Angle Left	Curve Elements R	T	C	B
1	Sta 26 +0.33	29°		600	155.17	303.69	19.74
2	Sta 12 +22.0		74°	350	266.15	455.09	89.70
3	Sta 24 +74.44	69°30'		550	381.55	667.15	119.39

Fig. 5. Plan of route

cuttings set back in order to provide safe visibility, and the traffic speed is restricted.

However, excessively long straight stretches of road, through monotonous surroundings, fatigue the driver and the passengers, especially on long journeys. It is shown by practice that the periodic insertion of horizontal curves of modest curvature improves drivers' attention and promotes the safety of traffic. For locating the curve, the following geometrical elements should be ascertained: angle α, radius R, arc length $C = AED$, tangent T, and bisector $B = BE$.

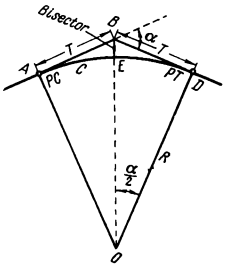


Fig. 6. Elements of a circular curve:

α—deflection angle; B—apex or intersection point; PC—point of commencement; PT—point of termination; R—radius; C—curve; T—tangent

Since during the investigation period the length of the route was measured along the tangents, a cumulative error arises in the overall measurement or chainage since the broken line ABD is longer than the arc AED (Fig. 6). In order to correct this error, one makes use of a *correction coefficient* λ for each curve when the length of the road is being measured.

The elements of the curve are interrelated by simple trigonometrical equations, which can easily be obtained from Fig. 6

$$\left.\begin{aligned} T &= R\tan\frac{\alpha}{2}; \quad BE = R\left(\sec\frac{\alpha}{2} - 1\right) \\ C &= \frac{\pi R\alpha}{180^\circ}; \quad \lambda = 2T - C \end{aligned}\right\} \qquad (1)$$

For the convenience of determining the length of curves and laying them out on the ground special tables are provided.

6. Elements of Road Profile

The section of a road made by a vertical plane along its centre line is termed a *profile.*

A profile shows the extent of *longitudinal gradients* of various road sections, and the relation of the level of the carriageway to existing ground level.

The rate of rise or fall of the longitudinal gradient is one of the most important characteristics of a motor road. In dry weather light passenger and freight vehicles, making use of their impetus, should be able to negotiate short stretches of road having a comparatively steep gradient (over 1 in 10).

In the case of combination vehicles, or where the road surface is dirty and slippery, the limiting negotiable gradient is appreciably gentler.

The natural land slopes often exceed permissible gradients for the effective use of motorized transport. In such cases the road gradient is made less steep than the slope of the ground by cutting into the shoulder of the rise or, alternatively, by forming embankments for the crossing of valleys or marshy ground.

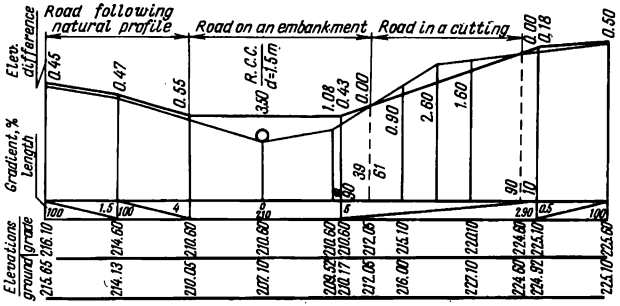


Fig. 7. Location of a road on an embankment, in a cutting and following the natural profile

When the road surface is situated below the land surface because the ground has been excavated, the road is said to be in a *cutting*. Places where the road is higher than the natural ground surface, i.e., where an artificially filled bed has been produced, are termed *embankments*. Because of the building of embankments and cuttings the road levels do not correspond to the ground surface levels (Fig. 7). The difference between the ground elevation and the grade elevation or formation line, which determines the height of the embankment to be filled in or the depth of the cutting to be excavated, is called the *working height* or *depth*, or *elevation difference* (Fig. 8).

The graphical representation of the profile is one of the main working drawings, on which the construction of the road is based.

The drawing of the profile has to conform strictly with established rules. Figure 9 shows an example of a drawing of a profile, as recommended in the U.S.S.R.

In order to accentuate the profile visually, the vertical intervals (levels) are drawn to a larger scale than the horizontal ones. For roads laid in a flat country the accepted vertical scale is 1 : 500 (5 m in 1 cm) and the horizontal scale is 1 : 5,000 (50 m in 1 cm).

The basic conventional symbols used for profiles are shown in Fig. 10.

For mountainous roads, where the profile is characterized by frequent alterations of land slopes and road gradients, and by an appreciable difference of levels within short road sections, it is customary to use larger scales, i.e., for vertical dimensions 1 : 200, and for horizontal ones 1 : 2,000.

The line on the profile which joins ground surface levels is called the *ground line.* The line which corresponds to the elevations

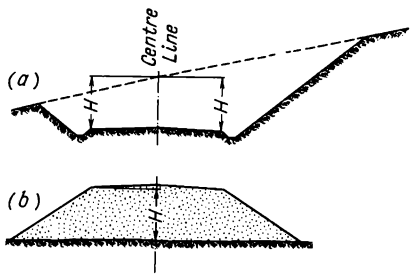


Fig. 8. Elevation difference of a roadbed in a cutting (*a*), and on an embankment (*b*)

of the roadbed verge is called the *formation line*, or *grade line.* On drawings the formation line is traced twice as thick as the ground line.

On the profile 2 cm below the ground line and parallel to it is drawn the *soil profile*, on which, by means of conventional symbols, (Fig. 11) are shown ground beddings in boreholes. For illustrating the soil profile a vertical scale of 1 : 50 (50 cm in 1 cm) is used.

7. Right-of-way and Road Cross-section

The zone which is marked for laying the road, excavating the soil for filling the embankments, for building ancillary structures and for green plantings is called the *road zone*, or *right-of-way.* The higher the technical classification of the road, the wider is the right-of-way for its construction.

Within the limits of inhabited places, preserves or agricultural lands used for especially valuable crops, the width of the zone should be reduced to a minimum, and include only the width strictly necessary for the road.

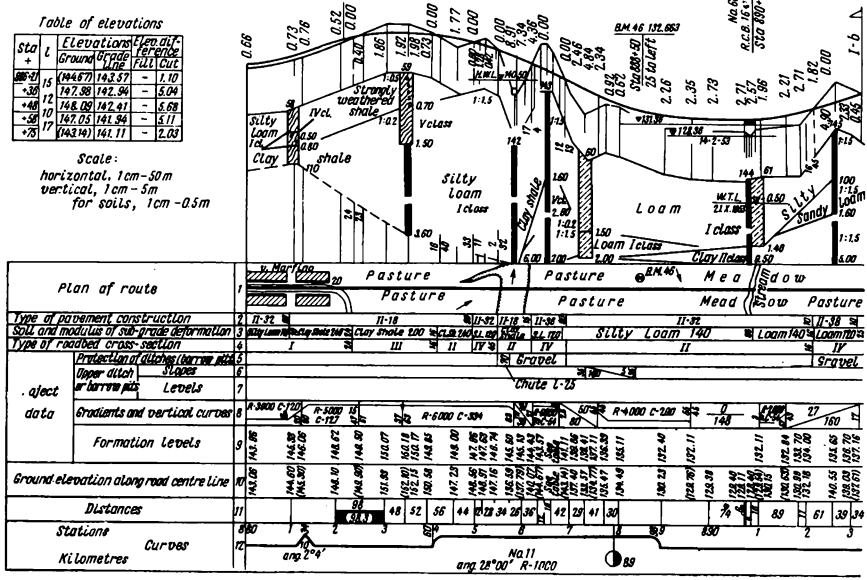


Fig. 9. Profile of highway section

Bench marks

B. M. 7-537 211

Sta 17+65 to right 30

B.M.8-563 217

Sta 16+00 to left 27

Slide roads

Slide road to the left at + 98 m, type of slide road II-a in both directions

Slide road to the right, type of slide road I-a to the left, I-b to the right

Level crossings

Unguarded crossing at + 27 m

Guarded crossing at + 83 m

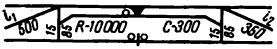
Drainage

Intercepting ditch 450 m long

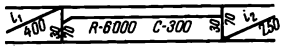
Discharge to left

Discharge to right

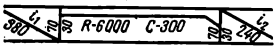
Vertical curves



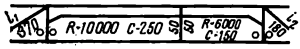
Convex vertical curve with rising and falling sections



Convex curve with rising section

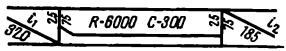


Convex curve with falling section



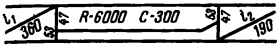
Connection at Sta +50 m of convex vertical curve with 10,000 m radius and convex curve with 6,000 m radius

Concave vertical curve with falling and rising sections



Concave curve with falling section

Fig. 10. Conventional



Concave curve with rising section

Connection at Sta + 10 m of convex curve with 6,000 m radius and concave curve with 3,000 m radius on a 3% gradient

Plan of alignment

Short station; the figures below indicate the station numbers

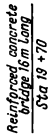
Curve with deflection angle to the right exceeding 5°

Curve with deflection angle to the left less than 5°

Kilometre according to actual kilometrage

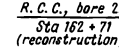
Kilometre according to design kilometrage

Road Structures

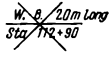


Projected structure

Existing structure



Structure to be reconstructed



Existing structure to be dismantled

Bridges

Wooden bridges

Minor bridges with slab girders

Deck truss bridges

Through truss bridges

Deck arch bridge

Through arch bridge

Overpass above designed road

Underpass below designed road

Culverts, etc.

Ovoid culvert

Round culvert

Box culvert

Channel

Wooden box culvert

Wooden triangular culvert

Filtering bank

Retaining Walls

Upstream retaining wall

Downstream retaining wall

Upstream revetment

Downstream revetment

symbols for profiles

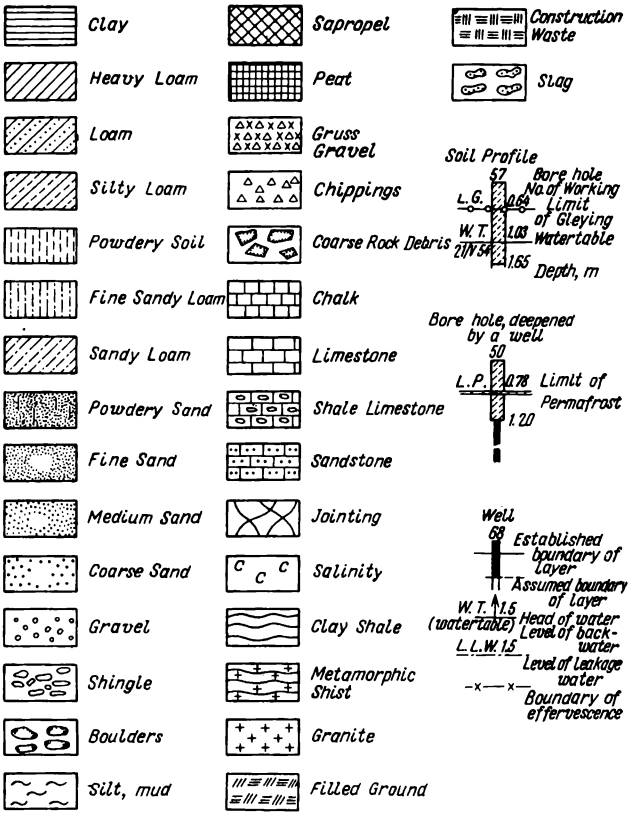


Fig. 11. Conventional symbols for marking soils on a profile

In separate cases, for example for building offices, for excavation of quarries for road-building materials, for planting fruit trees as protection against snowdrifts, excavation of borrow pits near the route, certain additional areas may be allocated. However, after the termination of construction works, the land which was appropriated for temporary structures, for the excavation of borrow pits and quarries, has to be levelled, and rendered suitable for further use in agriculture.

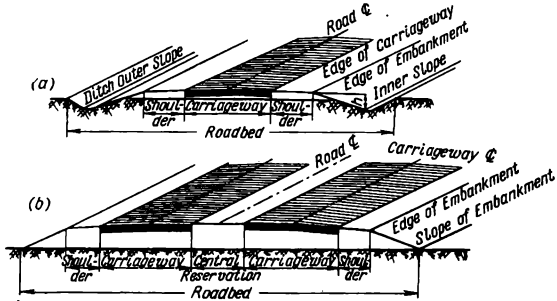


Fig. 12. Road cross-section:
a—single carriageway; *b*—dual carriageway with a central reservation

The section of a road by a vertical plane at right angles to the road centre line is called a *cross-section* (Fig. 12). The road surface strip within the limits of which motor vehicles run is called a *carriageway* or *roadway*. Usually it is reinforced by means of natural or artificial stone aggregates (concrete), which form the *pavement*. The strips of ground adjacent to the carriageway are called the *road shoulders*. The shoulders render lateral support to the pavement, which is made of solid materials within the limits of the carriageway. Shoulders are used for temporary parking of vehicles, as well as for road machinery during the road overhaul or for stacking road repair materials.

In order to lay the carriageway at the required level above the ground surface a *formation* or *roadbed* is constructed in the form of embankments or cuttings with side ditches for drainage and the diversion of water. The formation includes also *borrow pits*—shallow excavations from which the soil was used for filling the embankments, and *spoil banks*, which are heaps of excessive soil, parallel to the road, remaining after the excavation of cuttings.

The carriageway and shoulders are separated from the neighbouring land by properly battered faces or slopes. The cuttings and side ditches have *inner* and *outer* slopes. The junction of the surfaces of the shoulders and the embankment slope, or the ditch inner slope, is called the *edge* of the roadbed. The distance between the edges is called the *width of the roadbed.*

Cross-sections of a road with embankments are shown in Fig. 13. If the embankment is low and enough soil for its construction can be taken from the slightly widened side ditches, one says that the road is laid to follow the natural profile (Fig. 13*a* and *b*). When the embankment is high the soil has to be borrowed from cuttings adjacent to the road or taken from shallow excavations made near the road and called *borrow pits.*

The dimensions of the borrow pits are determined according to the quantity of the soil required for filling the embankment. The depth of the borrow pits should not exceed 1.5 m or be less than 0.3 m. Depending on local conditions, these excavations can be made on one or both sides of the road.

The borrow pits should be excavated as near as possible to the road because by reducing the length of haul of the soil from the pit to the embankment one reduces the cost of the earthworks and improves the utilization of road machinery. Therefore, when the constructed embankments are not high the borrow pit is often combined with the side ditch (Fig. 13*c*). The distance from the edge of the embankment to the bottom of the pit (the total height of the embankment and the depth of the pit) should be a maximum of 4 m. The width of such a combined ditch and borrow pit is determined according to the quantity of required soil, but it should not be deep and narrow.

When the embankments are high and require a great quantity of soil, the borrow pits are usually excavated away from the embankment. The strip of earth between the edge of the pit and the foot of the embankment slope is called a *berm.* The minimum width of a berm should be 2 metres and should vary according to the height of the embankment. Berms increase the stability of high embankments and during the construction period of the embankment can be used for the circulation of road machinery and cars, since this is more convenient than using the irregular floor of the borrow pit. The berm is given a transversal inclination of 2 per cent towards the borrow pit for drainage purposes.

As far as possible the width of the borrow pits and of the combined ditches and pits should be constant for a considerable distance, since frequent alterations in width mar the appearance of the road. The borrow pits should be thoroughly levelled during the finishing operations.

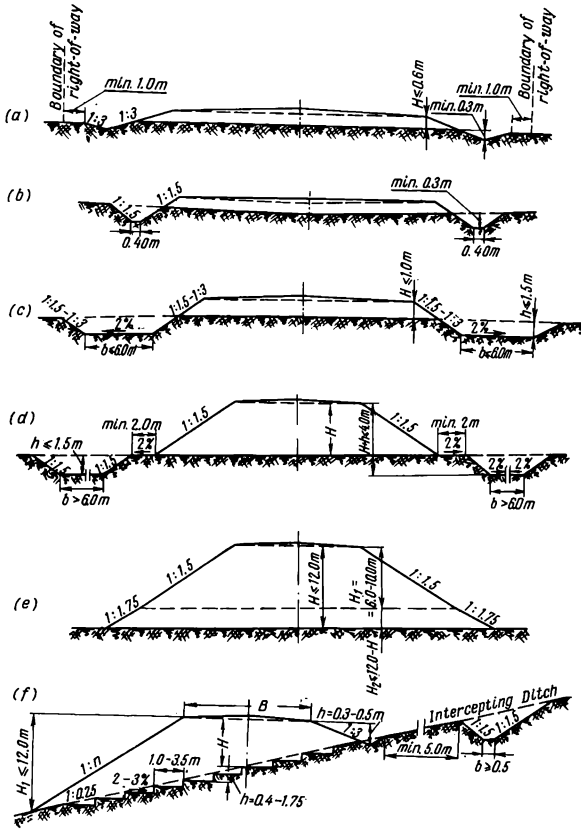


Fig. 13. Cross-sections of roadbed embankments:
a—with triangular side ditches; *b*—with trapezoidal side ditches; *c*—with ditch-borrow pits; *d*—with borrow pits; *e*—high embankment; *f*—on hillside

When planning the earthworks one should try to use earth excavated from cuttings or from the levelling of ground irregularities within land not being used for agriculture, rather than borrow soil from pits. The sides of the embankments are battered to form regular slopes. The slope gradient is specified by the slope ratio, i.e., by the ratio of the height of the slope to its horizontal projection.

The slopes of small embankments, with maximum heights of 1 m, are made with a ratio of 1 : 3* or less, in order to enable vehicles to drive off the road in an emergency.

Embankments having a height of more than 1 m, and the embankments at approaches to bridges, at fluvial plains, marshes and in other places where there is no possibility of diversion off the road, are formed with steep side slopes, of 1 : 1.5. This applies to embankments up to a maximum height of 6 m. Long experience proves that such embankments are quite stable. Steeper slopes of high embankments, however, may fail under the influence of their own weight, or the weight of a vehicle stationed on the shoulder, when the soil is saturated.

To ensure the stability of higher embankments the foot of the slope is made less steep (1 : 1.75). The depth of the above section of the embankment having the ratio of 1 : 1.5 is taken as 6 m in clay, loamy and silty ground; 7 m in sandy loam and fine sand; 9 m in medium and coarse sand; and 10 m in gravelly, gritty and soft, easily weathered rocky ground. The gradients of the inner slopes of the borrow pits are made the same as those of the embankments for which the borrowed material is being used.

The cross-section of a cutting in flat country is shown in Fig. 14*a*. If the soil excavated from the cutting is not suitable as fill material and if there is no practical reason to haul it to the nearby fills, then it may be used primarily for filling depressions of the site, or for easing the embankment slopes. If no other use of the soil is practical, it may be heaped at the side of the road, parallel to the edge of the cutting, into spoil banks which must be given a proper geometrical shaping.

The maximum height of a spoil bank should be 3 m; and the slopes facing the road should not exceed 1 : 1.5. The minimum distance of the spoil banks from the external edge of the slope of the cutting must be 3 m. If the cutting is made in water-bearing soil and spring water can seep from its slopes, the weight of the banked up soil may cause slips to develop. To prevent this, in soft and wet ground the spoil banks are placed at a minimum distance of $H + 5$ m from the edge of the cutting, where H is the depth of the cutting in metres.

* This denotes a slope of 1 vertical to 3 horizontal.—*Tr.*

In order to keep rain water from draining into the cutting from the spoil bank and the strip flanking the top of the slope, soil is heaped upon this strip and its section is given a regular triangular form; this is known as *benching*. The maximum height of benching

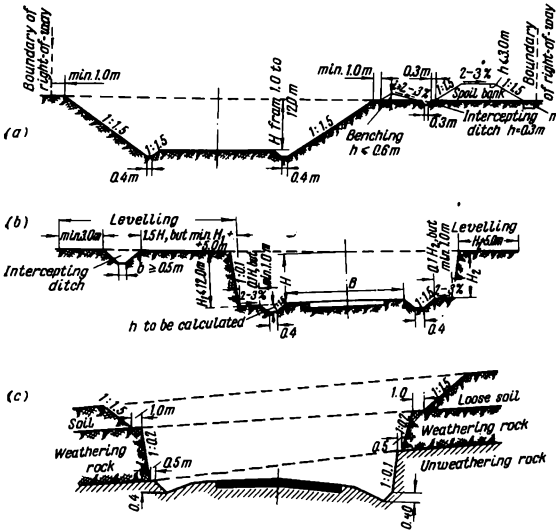


Fig. 14. Cross-sections of cuttings:
a—with spoil banks; b—in dry loessial soil; c—in stratified soil

is 0.6 m; the minimum distance of its toe from the edge of the cutting is 1 m, and the benching surface is trimmed down to a slope of two per cent away from the cutting. An *intercepting ditch* with a maximum depth and base width of 0.3 m is excavated between the benching and the spoil bank. To drain the water from the intercepting ditches, gaps are made in the spoil banks every 50 to 100 m when they are on the downgrade side of the cutting. On the upgrade side, water from the intercepting ditches is drained away at the lower end of the cutting.

Since the soil on the slopes of cuttings is more subject to saturation than on the slopes of embankments, faces of cuttings excavated in unconsolidated deposits should have a maximum gradient of 1 : 1.5.

In gravels and loamy soils, the faces of cuttings and embankments may be made with a slope of 1 : 0.5 to 1 : 1, depending upon the granulometric composition and density of the soil. Weathered rock may be trimmed down to a slope of from 1 : 0.5 to 1 : 0.2, according to the degree of weathering, the characteristics of the rock and the depth of the cutting. At the same time it is necessary to take account of the dip of the stratified layers outcropping on the slope, the resistance of the rock to weathering and the exposure of the

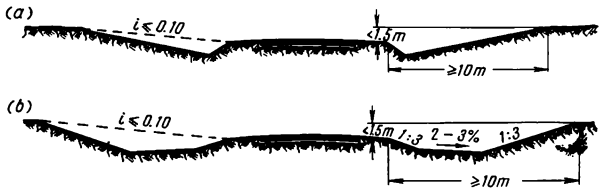


Fig. 15. Cross-sections of shallow cuttings in regions liable to snowdrifts: *a*—open cutting; *b*—cutting with subsidiary embankment

slopes of the cutting. Rocks such as shale and chalk which, when first excavated, seem quite stable become subject to intensive disintegration and weathering on exposure and crumble badly where the gradient is steep.

In countries with a dry climate vertical cuts can be made in loessial soils because of their special structure (thin vertical channels cemented with calcite) (Fig. 14*b*). Hence cuttings in loesses may be made with faces as steep as 1 : 0.1. However, steady spalling of loessial slopes occurs under the action of rain and wind. In order that the weathered materials do not choke the road ditch, a berm is constructed between the foot of the slope and the external edge of the ditch of a minimum width of 0.5 m, from which the weathered material is periodically removed. Such a cross-section, however, cannot be used in loess and loessial loams, neither can it be used in regions having a humid or rainy climate.

If the cutting is made through heterogeneous strata, its face may be given a broken or stepped contour (Fig. 14*c*). However, such a profile is aesthetically most unattractive, its formation is rendered difficult and, therefore, it is only permissible in cases where this would result in an appreciable reduction in the cost of the earthworks. When designing formations for construction in regions subject to heavy snowfalls, one must take into account their ability to withstand snowdrifts (Fig. 15).

PART II

Traffic Requirements to the Geometric Design of Highways

CHAPTER 3

TRACTIVE EFFORT AND PERFORMANCE OF VEHICLES

8. Movement of a Vehicle on a Road

All elements of a modern motor road should be designed to promote the safe passage of motor vehicles proceeding at the maximum rated speed (design speed).

The higher the traffic flow, the greater becomes the degree of restriction to the movement of individual vehicles, causing a reduction of their speed. Therefore, the requirements which are to be satisfied by the various elements of the road plan and profile, i.e., vertical and horizontal curve radii, the width of the carriageway, etc., must be based on the conditions of movement of a single motor vehicle travelling at a specific speed.

A moving motor vehicle is subjected to an exceedingly complex system of motions and forces, i.e., forward motion in a straight line, rotation about a vertical axis when driving around bends, vertical and horizontal vibrations caused by road irregularities, etc. Not all these traffic problems can yet be fully taken into account when defining the requirements with which the road is to comply. Because of this, when designing a road one assumes that the vehicle is moving without vibrations along an absolutely smooth hard surface which is not subject to deformation.

9. Resistance to Motion of a Vehicle

The tractive effort developed by a motor vehicle engine is used to overcome the forces resisting its propulsion.

Taking a general example of acceleration up a gradient, the following resistance forces are acting on the vehicle (Fig. 16): the

rolling resistance (rolling friction) P_f, the *resistance to motion up a gradient* P_i, *air resistance* P_w, *inertia forces* P_j which occur when the speed is altered.

The forces of rolling resistance and air resistance are always acting on any vehicle in motion. Those of the resistance due to the gradient and the forces of inertia depend on the character of the road profile and on the vehicle operating conditions. These forces may be absent, or even have a negative value assisting the movement (e.g., movement downgrade).

Rolling resistance is caused by shocks and impacts when the wheels of the vehicle run over the irregularities of the road surface,

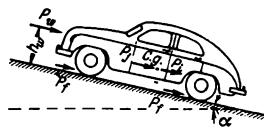


Fig. 16. Forces acting on a vehicle on an inclined plane

by the loss of power caused by the deformation of pneumatic tyres and by plastic and elastic deformations of the road pavement.

When the movement takes place along metalled roads on which no ruts are formed, the rolling resistance is directly proportional to the *rolling load*

$$P_f = \sum G_i f_i \tag{2}$$

where G_i = rolling load of individual wheels
f_i = corresponding factor of rolling resistance.

When the movement causes the formation of ruts, a more complicated relation occurs between the vehicle rolling resistance and the wheel load

$$P_f = \xi \sqrt{\frac{H}{D}} \cdot G_i \tag{3}$$

in which H = depth of the rut after passing of the wheel
D = wheel diameter
ξ = coefficient which varies from 0.6 to 1 depending on the state of the ground.

Usually the factor of rolling resistance is assumed to be directly related to the total weight of the motor vehicle, i.e., one assumes

$$f = \frac{\sum P_f}{G} \tag{4}$$

where $\sum P_f$ = total rolling resistance of all the wheels of the vehicle
G = weight of the vehicle.

The factors of rolling resistance for wheels with pneumatic tyres running on various surfacings have the following values:

	Factor of rolling resistance
Cement concrete and asphalt surfacing	0.01 to 0.02
Road with smooth chippings or gravel surface, treated with bituminous binders	0.02 to 0.025
Chippings or gravel surfacings, not treated with binder, having small pot-holes	0.03 to 0.04
Cobblestone pavement	0.04 to 0.05
Earth road, smooth, dry and compact	0.03 to 0.06
Ploughed field, saturated and swampy ground, loose sand	0.15 to 0.30 and over

The rolling resistance upon various surfaces is attributable to different reasons. On smooth cement concrete and asphalt surfacings the main cause of resistance to motion is the deformation of tyres. On less smooth surfacings—chippings, gravel and cobblestone pavements, additional rolling resistance is caused by the absorption of power in deforming springs and shock absorbers when impacts and jolts occur due to the surface irregularities. When traversing an irregular and muddy road, additional resistance is created by the compaction and extrusion of soil in the formation of ruts, the sticking of soil to the wheels, the spinning of wheels, impacts and jolts due to the vehicle swaying on bumps.

The rolling resistance depends on the speed of the vehicle and the elasticity of the tyres. Each time a wheel runs over a road surface irregularity a shock occurs which causes a loss of wheel velocity. To propel the vehicle with constant speed on an irregular surface, additional power is required. Since the magnitude of jolts in this case is proportional to the square of the rolling velocity, the factor of rolling resistance increases with the speed of the vehicle. When the speed exceeds a certain critical value the tyres develop radial vibrations, and the factor of rolling resistance increases sharply. The lower the air pressure in the tyres, the less is the speed at which this effect takes place. Nevertheless, one can assume that up to the speeds of the order of 50 km/hr the value of the factor of rolling resistance remains practically constant (Fig. 17).

When calculating tractive efforts corresponding to motor vehicles moving at a speed approaching the design speed of the road, i.e., in the range of 50 to 150 km/hr, one should take account of the increase of the factor of rolling resistance as expressed by the following formula:

$$f_V = f_0 [1 + 0.01 (V - 50)] \tag{5}$$

where V is the speed in km/hr.

In calculations related to the traversing of a road by high-speed motor cars, for instance racing cars, it is necessary to refer to the values of this factor established by relevant experiments on high-speed tracks.

Air resistance is caused by:

(1) reaction pressure of the air against the front of the vehicle;
(2) the friction of the air against the sides of the car body;
(3) power loss caused by eddying of the air stream behind the vehicle, under the body and around the wheels.

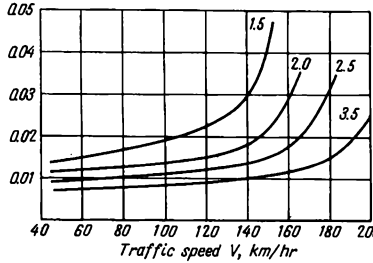


Fig. 17. Relation between the factor of rolling resistance and traffic speeds for various tyre pressure (the figures next to the curves give the tyre pressure in kg/cm²)

According to the laws of aerodynamics, the air resistance to automobile motion is expressed by the following relation:

$$P_w = \frac{c\rho\omega V^2}{13} \tag{6}$$

where c = factor of ambient air resistance (a dimensionless factor, depending on the shape of the body moving in the air, and also on the smoothness of its surface)

ρ = density of air, which at sea level is 0.125 kg·sec²/m⁴

ω = area in square metres of the projection of the vehicle on a plane at right angles to the direction of its movement (frontal projection); for modern vehicles it can be obtained by the following formula: $\omega = 0.775\ BH$, where B and H are overall width and height of the motor vehicles

V = motor vehicle speed relative to the ambient air, km/hr; with following wind $V = (V_{veh} - V_{wind})$; with head wind $V = (V_{veh} + V_{wind})$.

TABLE 2

Type of vehicle	Coefficient of air resistance K, kg·sec²/m⁴	Area of frontal projection ω, m²
Truck	0.05 to 0.07	3 to 6
Single-deck coach	0.025 to 0.05	4 to 6.5
Semi-streamlined passenger car	0.015 to 0.03	1.5 to 2.6
Streamlined racing car	0.010 to 0.015	1.5 to 2.0

For automobile calculations the product $c\rho$ is replaced by the *coefficient of air resistance* K, which is determined experimentally (Table 2).

Customarily, time-speed-distance calculations are made assuming calm windless weather conditions.

Air resistance increases sharply with increase of traffic speed and is the main type of resistance encountered by moving passenger cars. Because of this, when designing motor cars, substantial consideration is given to the possibility of reducing air resistance by means of improving the streamlining of the body.

The contour of a vehicle has a great influence on the air resistance. By means of comparatively simple alterations in the shape of truck bodies one can reduce the value of the air resistance factor and, therefore, increase the economic and dynamic characteristics of the vehicle (Fig. 18). It must be mentioned that arrangements which lower the resistance to motion of a solo vehicle may adversely influence the streamlining when the vehicle is used as the tractor of a combination vehicle.

The resistance to motion when driving a vehicle up an incline with a slope i is increased as the result of the additional work done in propelling the vehicle up the slope.

If the length of the slope is L and the height gained as the result of travelling up the slope is H, then the additional work needed for propelling a vehicle of weight G up the slope can be expressed as

$$F = GH$$

Ignoring the difference between the actual length of the inclined section and its horizontal projection, which will be negligible in the case of longitudinal gradients on a motor road, the additional work done in travelling a unit distance up the slope is

$$f_i = \frac{F}{L} = \frac{GH}{L} = Gi \tag{7}$$

Should the incline or slope i be expressed in per cent, then formula (7) will become

$$f_i = G\frac{i}{100} \tag{7a}$$

Thus the work done in travelling unit distance up the slope is the product of the total weight and the value of the tangent of the slope expressed as a decimal fraction.

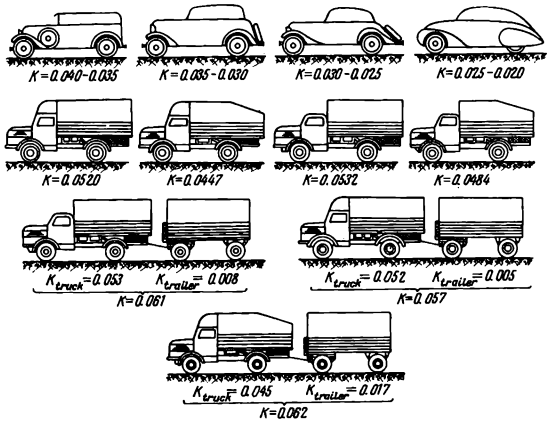


Fig. 18. Influence of the aerodynamic shape of a vehicle on the factor of air resistance

The resistance of the inertia forces of a vehicle is made up of the inertia of the forward motion and the inertia of the revolving parts of the vehicle. These forces act on the vehicle during its acceleration or deceleration.

If the mass of the vehicle is $m = G/g$, then the inertia force of the forward motion is

$$P'_j = m\frac{dv}{dt} = \frac{G}{g}\frac{dv}{dt} = Gj \tag{8}$$

where dv/dt = vehicle acceleration

$j = \frac{1}{g}\frac{dv}{dt}$ = acceleration relative to gravitational.

With the alteration of the vehicle speed, one must add to the inertia of the forward motion the inertia of the vehicle revolving parts (wheels, flywheel and transmission gears).

The exact value of the inertia forces can be calculated according to the dimensions and weight of the revolving parts. However, in practice, to take into account the influence of the revolving parts, the forward-motion inertia is usually used with a correction factor β, which gives the ratio between the total force necessary to give the impetus to all the vehicle parts having translational and rotational motion, and the force required exlusively for the impetus of masses in translation

$$\beta = \frac{P_{tra} + P_{rot}}{P_{tra}} \tag{9}$$

With the increase of the gearbox reduction ratio the value of the factor β increases. With a direct drive the value of the factor β varies between 1.03 and 1.07. Using other gears, its value increases approximately as follows:

$$\beta = 1.04 + n i_k^2 \tag{10}$$

where i_k = gearbox reduction ratio
n = factor equal to 0.03-0.05 for passenger cars and to 0.05-0.07 for freight vehicles.

When a vehicle is proceeding with a frequently varying speed and with frequent stops, for example in urban conditions, apart from the inertia of the revolving parts a peculiar engine thermal inertia arises. For established operating conditions of an engine proceeding with a constant speed on a smooth horizontal sector there are corresponding constant conditions of engine temperature and of fuel admission and combustion. When, on the other hand, the speed varies, the engine cannot adapt itself immediately to the alterations of loading and therefore the combustion process in the engine takes place under less advantageous conditions than with constant operating conditions. This causes a certain decrease in the available tractive effort.

The influence of unstable engine operating conditions on the dynamics of the vehicle can be taken into account, as in the case of revolving engine parts, by introducing a correction factor δ in the expression for the inertia of masses having a translatory movement.

The additional inertia resistance due to the influence of unstable engine operating conditions is therefore

$$P_j'' = \delta i_k^2 \tag{11}$$

The value of the factor δ for trucks is 0.07-0.075, and for pas enger cars it is 0.075-0.085.

Thus, the general expression of the resistance of inertia forces to the vehicle movement is as follows:

$$P_j = G_j [1.04 + (n + \delta) i_k^2] \tag{12}$$

10. Dynamic Characteristics of a Vehicle

The mechanical power generated by the vehicle engine is transferred by the transmission of the vehicle to its driving wheels. At the area of contact of the wheel with the road the wheel torque M_w induces a tractive force P_p equal to

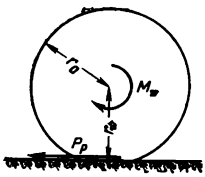


Fig. 19. Driving wheel tractive effort

$$P_p = \frac{M_w}{r_w} \tag{13}$$

where $r_w = \lambda r_0$ is the radius of the wheel rolling circle with account taken of the tyre deformation (Fig. 19).

The tyre deformation factor λ on a hard surface will attain 0.945-0.950 for high-pressure air tyres and 0.930-0.935 for low-pressure tyres.

The driving wheel torque M_w is the product of the engine torque M_b, the gear ratios of the transmission and the mechanical efficiency

$$M_w = M_b i_g i_m \eta \tag{14}$$

where i_g = gearbox transmission ratio
i_m = main drive transmission ratio
η = mechanical efficiency of the vehicle transmission, which takes account of the power loss due to overcoming the resistance of all mechanisms between the engine and the driving wheels.

The tractive effort P_p is therefore

$$P_p = \frac{M_b i_g i_m \eta}{r_w} \tag{15}$$

But the engine torque is proportional to the engine horsepower N_b and inversely proportional to the number of revolutions of the crankshaft n, hence $M_b = N_b/n$, and formula (15) will transform into the expression

$$P_p = 716.2 \frac{N_b i_g i_m}{n r_w} \eta \tag{16}$$

where N_b is the engine brake horsepower.

The speed of the vehicle is related to the crankshaft speed, as follows:

$$v = \frac{2\pi r_w n}{60 i_g i_m} \text{ m/sec}$$

or

$$V = \frac{0.377 r_w n}{i_m i_g} \text{ km/hr} \tag{17}$$

where n = crankshaft speed, rpm
i_g and i_m = transmission ratios of the gearbox and of the main drive.

The maximum vehicle speed corresponds to the minimum tractive effort. Therefore, when a vehicle is moving along a good road, where the resistance to rolling is small, the direct drive will tend to be used ($i_k = 1$). When negotiating a gradient or driving along a bad road, one will need to change down to a lower gear.

The horsepower produced depends on the rotational speed of the engine. This relation is usually determined experimentally and represented diagrammatically by *external power-speed characteristics* (Fig. 20).

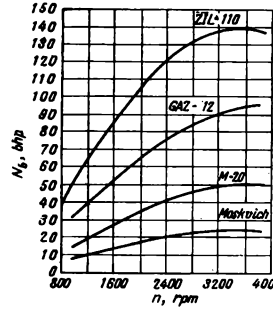


Fig. 20. External pover-speed characteristics of engines

The vehicle external power-speed characteristics are usually obtained by testing the vehicle at full throttle for engines fitted with carburettors or, in the case of fuel injection, at maximum fuel pump delivery. Therefore, these curves specify the maximum power which the engine can develop.

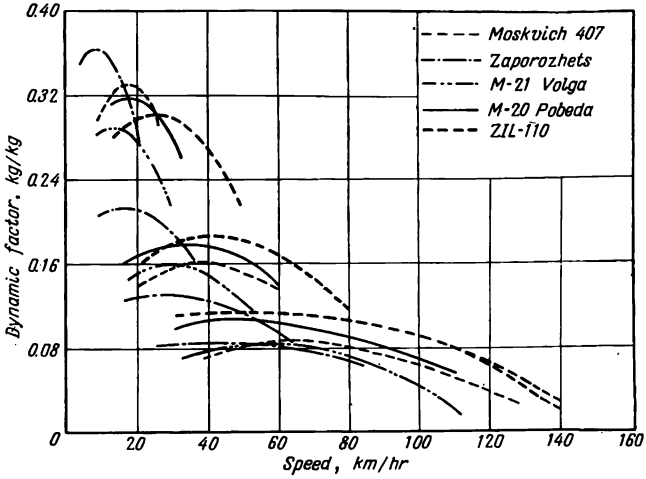
Using this characteristic, and formulas (6) and (16), one can determine by calculation the tractive effort P_p at various vehicle speeds, assuming that it is utilized in overcoming the resistance to propulsion.

Equating external and internal forces, the following expression is obtained:

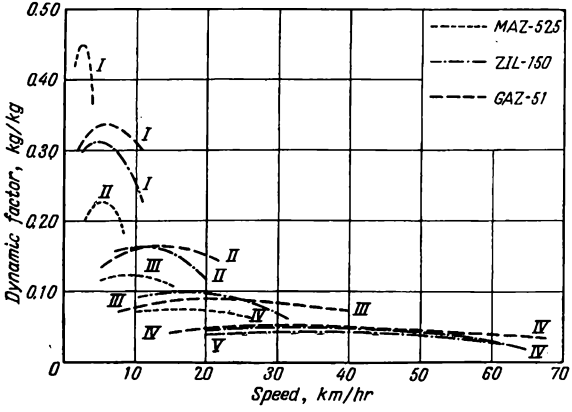
$$P_p = P_f \pm P_i + P_w \pm P_j \tag{18}$$

where P_f = rolling resistance
P_i = resistance to motion up (+) or down (—) a gradient i
P_w = air resistance to movement
P_j = resistance of inertia forces, the $\pm$ sign indicating that, depending on the correlation of the external resistance forces, the vehicle may be either accelerating or decelerating.

Transferring the air resistance, which depends on the speed of the vehicle, to the left side of the equation and substituting the



(a)



(b)

Fig. 21. Motor vehicle dynamic characteristics:
a—passenger cars; b—commercial vehicles

values of the resistances as determined in Sec. 9, we obtain

$$P_p - K\omega v^2 = Gf \pm Gi \pm Gj \tag{19}$$

The Soviet Academician E. A. Chudakov proposed to define the tractive, or dynamic, characteristics of a motor vehicle by means of a *dynamic factor*, which is the difference between the full tractive effort on the driving wheels and the air resistance, per unit weight of the vehicle

$$D = \frac{P_p - K\omega v^2}{G} = f \pm i \pm j \tag{20}$$

The dynamic factor gives the surplus of the tractive effort per unit weight of a vehicle moving with a speed v, which can be used for overcoming road resistance ($f \pm i$) and for imparting to the vehicle the acceleration j.

The tractive effort and the air resistance depend on the speed of the vehicle. Therefore, the value of the dynamic factor does not remain constant but is varied with change of speed. The diagram showing the relation between the dynamic factor and the speed, known as a *dynamic characteristic* (Fig. 21), is used in the Soviet Union as a basic index of automobile traction performance and underlies all motor road time-speed-distance calculations.

11. Adhesion of Pneumatic Tyres to the Road Surface

The driving wheel tractive effort may be developed only if there is sufficient adhesion between the driving wheels and the road. The ratio of the tractive effort generated at the contact face P_p to the vertical load on the wheel G_w at which the wheel starts slipping (spinning), is called the *coefficient of adhesion* and is designated by the symbol φ.

To improve the conditions of adhesion between the tyre and the road surface and to expel water more effectively when running on a wet road, the tyre contact surface is grooved creating a salient pattern—a tread.

When determining the desirable characteristics of the road, two separate coefficients of adhesion need to be taken into account:

1. The coefficient of adhesion corresponding to the commencement of wheel spin without the development of any lateral force—this is the *coefficient of linear adhesion.*

2. The coefficient of adhesion introduced when the wheel moves at an angle to the plane of rotation, i.e., it is simultaneously spinning and moving parallel to its axis—this is the *coefficient of lateral adhesion.*

Since the tyre resilience is not the same in the plane of rotation as in the lateral direction, the value of the coefficient of adhesion alters with the tyre deformation in various directions.

However, for estimates relating to the road design, it is usual to assume that when a force acts on the wheel at an angle to the direction of its rotation the absolute value of the coefficient of adhesion is that of the coefficient of linear adhesion.

Numerous experimental investigations to determine the coefficient of adhesion have shown that its value is influenced more by the condition of the road surface than by its type. This is because with any surfacing, the protruding hard mineral parts press into the rubber of the tread and, therefore, the wheel slips mainly owing to the deformation of the rubber.

With continual wear and tear the roughness of the pavement decreases and with it there is a decrease in the adhesion properties of the wheel.

In wet and muddy conditions the interstices in the pavement surface become filled with dirt, dust, etc. A moist film acts as a lubricant between the rough surfaces, wettening the area of contact between the tyre and the tread, hence decreasing the value of the coefficient of adhesion. At high traffic speeds the tyre is not given sufficient time to deform fully and therefore the surface irregularities press into the tyre to a lesser depth causing a reduction in the effective coefficient of adhesion. This effect is particularly apparent on wet and dirty surfacings. On dry surfacings this reduction of the coefficient of adhesion is less noticeable.

The relation of the safe, attainable coefficient of adhesion to the forward speed on a wet cement concrete pavement as suggested by the International Association of Civil Aviation for the design of airfield runways, is shown in Fig. 22.

Since during the braking action the speed of the vehicle varies over a wide range, calculations for the safe stopping distance are based on the values of the coefficient of adhesion at speeds of the order of 30 to 40 km/hr, which are the mean speeds for the whole braking process. The values of the coefficient of linear adhesion in relation to the state of the pavement are as follows:

Nature of road surface	Coefficient of linear adhesion
Dry, rough-surfaced (cement concrete)	1 to 0.7
Dry, smooth (bituminous surfacing)	0.5
Wet	0.4 to 0.3
Muddy	0.2
Ice covered	0.1

The conditions of pneumatic tyre adhesion to the road pavement are related to the weather conditions, and the value of the coefficient of adhesion fluctuates greatly during the year.

In summer it increases markedly. In winter, however, when ice covers the road, the coefficient is much lower, and to increase adhesion it is often necessary to fit antiskid chains to the wheels or spread sand or grit over the road surface. Therefore, when estimating the coefficient of adhesion for time-speed-distance calculations during road design, one has to consider the climatic conditions of the region. Depending on the ratio of summer sunny and rainy days, the length of the period of snow covering, and the frequency of occurrence

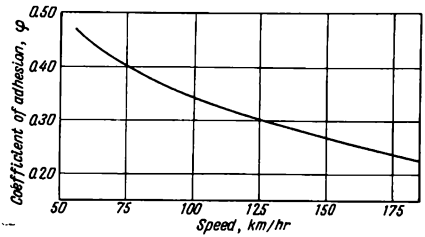


Fig. 22. Relation between the coefficient of adhesion and aircraft speed over a wet cement concrete surfacing

of ice covering, a most typical condition of the pavement surface is chosen, which is then used for time-speed-distance calculations. Also, one has to consider the seasonal distribution of traffic, so that, in the case of concentration at definite periods (e.g., seasonal agricultural transportations), a maximum safety of traffic can be assured. On the other hand, there is no necessity to base the design on the pavement conditions at certain infrequent and short periods when the weather conditions might deteriorate, since special precautions can then be taken by the road service authorities with the introduction of temporary speed restrictions.

The coefficient of adhesion between the tyre and the surfacing is one of the main traffic safety characteristics for a road, since, as it is shown below, the extent of the stopping distance is inversely proportional to the value of the coefficient of adhesion. The statistics of accidents on roads show that the periods at which the value φ decreases owing to weather conditions correspond to the increase of the number of accidents (Fig. 23). In view of this, in a number of countries the road maintenance organizations systematically check the variation of the coefficient of adhesion, and when its value decreases below a certain minimum (usually $\varphi = 0.4$-0.5) have the duty to display signs limiting the traffic speed, if this decrease has a tem-

porary character due to weather conditions, or take the necessary precautions to increase the roughness of the surface—usually by means of some surface treatment.

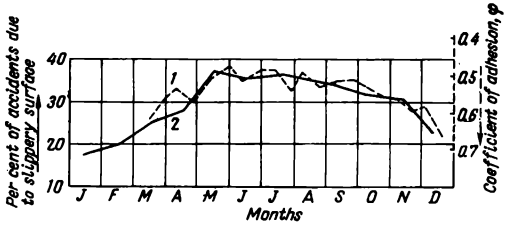


Fig. 23. Relation between the value of the coefficient of adhesion and the number of accidents:
1—coefficient of adhesion; 2—number of accidents

The conditions of adhesion of the driving wheels with the road surface limit the dynamic capabilities attainable in a motor car,

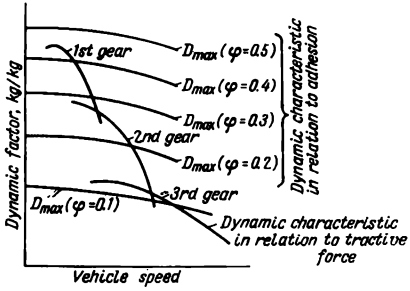


Fig. 24. Vehicle dynamic characteristic in relation to adhesion

since with a poor coefficient of adhesion, the great tractive effort which the engine can develop cannot be used owing to insufficient adhesion.

Because of this, besides the engine horsepower dynamic characteristics, adhesion dynamic characteristics are used for time-speed-

distance calculations. The adhesion characteristics are deduced from the tractive effort equation for motion without slipping

$$G'\varphi = Gf \pm Gi \pm Gj + K\omega v^2 \tag{21}$$

where G' is the driving wheel pressure on the road.

Relating the balance of adhesion available over the ambient air resistance to a unit of vehicle weight, the expression of the adhesion dynamic characteristic becomes

$$D_a = \frac{G'\varphi - K\omega v^2}{G} = f \pm i \pm j \tag{22}$$

An example of a dynamic characteristic chart related to the conditions of adhesion is shown in Fig. 24.

12. Longitudinal Gradients Negotiated by Motor Vehicles

The chart of dynamic characteristics permits the solution of problems concerning the motion of a motor vehicle when travelling along a road and enables the following to be determined:

(1) The magnitude of the longitudinal gradient which can be negotiated at a given constant speed.

To solve this problem with the help of the chart of dynamic characteristics, a vertical line is drawn from the abscissa corresponding to the given speed, to its intersection with the curve of dynamic characteristic (Fig. 25). The ordinate of this point gives the value of the dynamic factor D, which is equal to the sum

$$i + f + j$$

Since it is assumed that the motion takes place at constant speed, then $j = 0$, and, therefore,

$$i = D - f$$

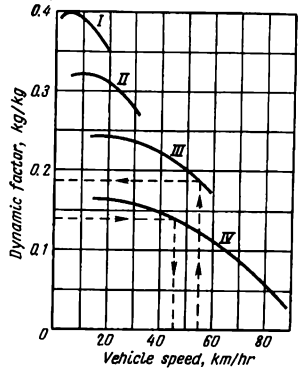


Fig. 25. The use of dynamic characteristics for time-speed-distance calculations. (Roman numerals denote the gear used.)

(2) The speed which a vehicle can maintain on a given gradient. The factor required for this condition is obtained from the relation

$$D_1 = i + f$$

The speed v is determined by measuring the above value on the ordinate axis and then finding the corresponding abscissa with the help of the chart of dynamic characteristics (Fig. 25).

(3) The acceleration developed by a vehicle under the influence of gravity.

With a rolling resistance factor f, a gradient i and an initial speed v, the linear acceleration of the vehicle becomes

$$\frac{\beta}{g} \cdot \frac{dv}{dt} = D - (f + i) \tag{23}$$

where β is a coefficient taking into account the influence of the inertia of the revolving parts of the vehicle.

Since the solutions described above are based on the value of the tractive effort developed by the engine, they have to be further checked in relation to adhesion. For this reason, the same diagram should contain curves expressing the dynamic characteristics in relation to adhesion.

If the point obtained according to the first estimate is situated below the curve of dynamic characteristics in relation to adhesion then the assumed tractive effort will not cause slipping and the estimate is correct.

The methods of assessment of tractive efforts given above relate to what are termed *equilibrium speeds*, which are constant within the limits of the gradient. However, the vehicle usually approaches the slope with a certain accumulated momentum, which can be used to overcome the additional resistance at the expense of a gradual diminution in speed. If the gradient exceeds the incline corresponding to the equilibrium speed, then the speed gradually falls. Provided that the speed of the vehicle within the limits of the gradient does not drop to zero, the vehicle can negotiate the upgrade.

The question of the effect of a succession of gradients on adjacent sections of a road upon motor traffic has not been fully investigated. Sufficient account is seldom taken of this issue when designing highways for motor vehicles. Nevertheless the rational coordination of gradients can contribute to the improvement both of road transport productivity and fuel economy, since it enables upgrades to be traversed with the aid of initial impetus and, within the limits of safety, reduces the use of brakes on downgrades.

If the initial momentum of vehicles is to be taken into account, the question arises as to what will be the length of an upgrade of slope i_{max} —where i_{max} is greater than the gradient i, the maximum gradient of infinite length negotiable by vehicles—if the speed of approach is V_1 and the speed at the end of the climb is not to be less than V_2.

Assuming a constant tractive effort or engine propelling force, the initial momentum is $\frac{\beta G}{2g} V_1^2$, and the terminal momentum $\frac{\beta G}{2g} V_2^2$. This difference in momentum is expended in overcoming the additional resistance to motion up the gradient $(i_{max} - i)$.

The additional work done on a gradient of length L is $LG\,(i_{max} - i)$. Therefore

$$LG\,(i_{max} - i) = \frac{\beta G}{2g}\,(V_1^2 - V_2^2) \tag{24}$$

Hence, the maximum permissible length of road L having a gradient of $i_{max} > i$, is

$$L = \frac{\beta\,(V_1^2 - V_2^2)}{2g\,(i_{max} - i)} \tag{25}$$

Here, as previously, the factor β includes the inertial effects of the rotating masses.

The above calculation is only approximate since the ambient air resistance has been assumed constant. A more detailed computation, involving the solution of the differential equation governing the law of motion of the vehicle, is given in the section dealing with motion along vertical curves.

13. Motion of Motor Vehicle along a Curvilinear Profile

Stretches of modern automobile highways include various longitudinal gradients which are interconnected by means of vertical curves of large radii (see Sec. 64). In undulating country 50% or more of the total length of highways in the higher classes may consist of vertical curves.

When a vehicle is moving along a curved profile the longitudinal gradient is constantly varying, and with it, the speed of the vehicle. Thus the above conclusions regarding "equilibrium speeds" are only relative.

In the equation of motion along a curvilinear profile the longitudinal gradient must be considered variable (Fig. 26).

If the coordinate origin is placed at the point of the start of the vertical curve, then the gradient at a certain point A is as follows:

$$i = \alpha + \xi = \alpha + \frac{d}{ds} \cdot [f\,(y)] \tag{26}$$

where α = angle between the horizontal and the chord connecting the extreme points of the curve which also serves as one of the axes of coordinates

$\xi = \frac{d[f(y)]}{ds}$ = angle between the tangent to the vertical curve and the chord

$f(y)$ = equation of the vertical curve.

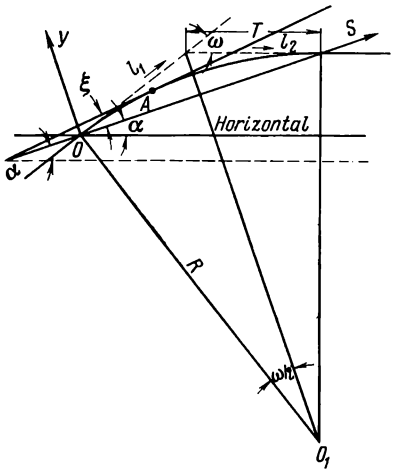


Fig. 26. Diagram illustrating the dynamics of motion of a vehicle traversing a vertical curve

For the case considered the equation of the dynamic characteristics is given by

$$\frac{P_p - P_w}{G} = f + \alpha + \frac{d[f(y)]}{ds} + \frac{\beta}{g} \cdot \frac{dv}{dt} \tag{27}$$

For the solution of the equation (27) Soviet engineer K. A. Khavkin assumed the vertical curve to be a parabola of the type

$$y = \frac{\omega}{2} S \mp \frac{1}{2R} S^2 \tag{28}$$

where ω = deflection angle, formed by the adjacent sections of the profile

S = abscissa of the curve, which may be assumed as about equal to the distance travelled by the vehicle along a vertical curve.

The equation of the dynamic factor is replaced by an empiric relation obtained from the equation (20) by substituting a parabola for the external characteristic curve

$$\frac{P_p - P_w}{G} = a - bv^2 \tag{29}$$

where v = speed, m/sec
a and b = indices characterizing the relation of the tractive effort and the traffic speed in various gears and at various throttle openings.

The values of the indices a, b and μ for some cars of Soviet make at full throttle opening are given in Table 3.

TABLE 3

Type of motor car	a	b	μ
GAZ-12	0.1096	0.52×10^{-4}	0.944×10^{-3}
M-20 Pobeda	0.100	0.765×10^{-4}	1.39×10^{-3}
GAZ-51	0.053	0.59×10^{-4}	1.091×10^{-3}
ZIL-150	0.055	0.73×10^{-4}	1.335×10^{-3}

The solution of the differential equation (27) gives a formula for determining the vehicle speed on various stretches of the vertical curve:

$$v_s = \sqrt{(v_i^2 - k_1)\, e^{-\mu s} + k_1 + k_2 S} \tag{30}$$

where v_i is the vehicle speed at the moment of entering the curve, m/sec

$$\mu = \frac{2gb}{\delta}$$

$$k_1 = \frac{1}{b}(a - f - i_1) - \frac{k_2}{\mu}$$

$$k_2 = \pm\frac{1}{Rb}$$

the plus sign corresponding to motion along a convex curve, and the minus sign to that along a concave one. By substituting $R = \infty$ and $k_2 = 0$, formula (30) may be used for determining the speed on a finite stretch of road with a uniform longitudinal gradient.

In Fig. 27 the results of speed calculations for the automobile GAZ-51 on upgrades with uniform inclinations are compared with those for a vertical curve with a radius of 10,000 m, assuming an initial velocity of 70 km/hr.

It is worth noting that for negotiating a uniform gradient it is necessary to change to a lower gear whilst the vertical curve enables the vehicle to climb the gradient in direct drive. This shows that

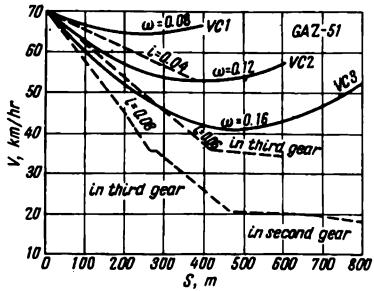


Fig. 27. The effect of the introduction of vertical curves on vehicle speed

the design of the profile with vertical curves not only increases the safety of traffic, but also improves the technical and economical characteristics of the road.

14. Braking and the Characteristics of Vehicular Motion on Downgrades

When determining the equation of motion of a vehicle down a gradient, the resistance due to the gradient is given a negative sign. Thus

$$\frac{P_p - P_w}{G} = f \pm j - i \tag{31}$$

This leads to an increase in the acceleration j, the vehicle will gather momentum, and its speed will rise rapidly.

The limiting vehicle speed down a slope is attained theoretically when the sum of ambient air and rolling resistances becomes equal to the combined force of traction and the momentum due to rolling down the incline, and the acceleration is reduced to zero

$$\frac{P_w}{G} + f = \frac{P_p}{G} + i \tag{32}$$

However, such a value would correspond to a very high vehicle speed and under practical conditions would be virtually impossible to attain. Driving a vehicle down a slope at high speed is dangerous,

especially on a rough surface when steering becomes difficult. Drivers may be forced to take special precautions depending upon the steepness of the gradient—throttling back, applying their brakes without declutching and making use of the engine for braking by engaging a low gear.

Braking is most effective with the clutch disengaged when the momentum of the vehicle is gradually dissipated by the friction of the brake shoes against the brake drum.

The equation which represents the movement of a vehicle with a disengaged clutch and an applied brake is as follows:

$$Gj = P_b + P_w \pm P_i + P_f \tag{33}$$

where P_w, P_i and P_f = resistances to motion
P_b = braking force, the magnitude of which is determined by the equation

$$P_b = \gamma_b G \tag{34}$$

where G = weight of the vehicle
γ_b = specific brake force factor, equal to the ratio between the sum of the braking forces acting on all the brake wheel rims and the weight of the vehicle.

The index γ_b is dependent on the number of brake wheels, on the condition and the adjustment of the brakes, and on the braking effort exercised by the driver, the latter depending on the purpose of braking and varying widely from gentle brake application to the complete locking of the wheels accompanied by skidding in an emergency stop.

Substituting in Eq. (33) the values of the resistances to motion we obtain the value of the negative acceleration

$$j = \frac{P_w}{G} + \gamma_b \pm i + f \tag{35}$$

Since during braking the speed of the vehicle decreases rapidly, and at speeds below 30 km/hr the air resistance is negligible, the influence of the latter on the braking process is usually ignored. If it is assumed that $P_w/G = 0$ the computed stopping distance, along which the brakes are applied, is increased by only 2-5%. However, when designing a road, such an increase tends to contribute to the safety of traffic.

The distance covered by the vehicle when reducing speed from v_1 to v_2 m/sec is

$$S = \frac{v_1^2 - v_2^2}{2g\gamma_b} \tag{36}$$

The speed of motor vehicles down a gradient as determined by the equation of motion (32) is very high and, in the interests of

safety, drivers cut down their speed. For time-speed-distance calculations on motor roads it is usually recommended that the downgrade speed assumed should correspond to the equilibrium speed required to negotiate an upgrade of an identical gradient. This statement concerns the case of passenger car motion, where there is a comparatively large reserve of tractive force. An investigation of the movement of heavy combination vehicles led to the conclusion that the speed on downgrades having a gradient over 4 per cent is about twice as high as the one used on an equivalent upgrade. This is caused by an insufficient magnitude of the dynamic factor.

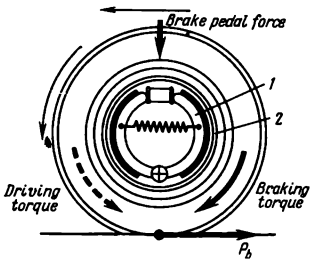


Fig. 28. Diagram showing operation of a motor vehicle brake:
1—brake shoe; 2—brake drum

In the process of braking, the driver, whose pressure on the brake pedal is transmitted by the brake linkage, creates a friction force between the brake shoes and the drum (Fig. 28).

If the engine is fully declutched, normal service brake application will correspond to a partial locking of the wheels which allows them to rotate with a certain amount of slipping. The braking force P_b will be the resistance P_τ, created by the braking torque, and the resistance of the tyre tread to slipping on the road P_φ.

As the result of the braking effort the wheel rotates with a velocity v which is lower than the speed of the vehicle v_0, v being equal to $v_0\lambda$, and the slipping speed of the tyre at the point of contact with the road is

$$v_{slip} = (v_0 - v) = v_0(1 - \lambda)$$

If one assumes that the effect of the brake action is directly proportional to the slip factor, then the total braking force is

$$P_b = P_\tau\lambda + \varphi G(1 - \lambda) \tag{37}$$

where $P_\tau = M_b/r_k$ = resistance created by the braking torque in the brake
M_b = braking torque
r_k = radius of the rolling circle
φ = coefficient of adhesion of the tyre to the surfacing when the wheels are completely locked.

With complete blocking of the wheels, $\lambda = 0$, and the vehicle will skid, i.e., $P_b = G\varphi$.

In practice, because of the heating of the tyre, the coefficient of adhesion is reduced and, as experiments have shown, the distance covered during braking which enables the wheel to rotate just short of slipping, is shorter than when skidding occurs.

The coefficient of specific braking force is

$$\gamma_b = \frac{P_b}{G} = \frac{\frac{M_b}{r_k}\lambda + \varphi G\,(1-\lambda)}{G} \tag{38}$$

The distance within which the driver can stop a vehicle from the design speed is an important safety factor, playing a vital role in determining the geometric standards for motor highways.

Equation (38), derived above, relates to a process of braking already in operation. However, there is a time lag between the moment when the driver sees the obstruction and the moment when the full braking action takes place. Hence in determining the safe stopping distance, it is necessary to consider the time required for the driver's reaction t_1, the delay in the operation of the brake drive Δt_1 and the nonuniformity of the braking force building up during the braking process t_2.

The time lag in the operation of the braking system is 0.03 sec for a hydraulic brake and 0.3 sec for an air-operated one. The period of the building up of the braking effort is 0.2 sec for the hydraulic gear and 0.4 to 1 sec for an air system.

The driver's reaction time t_1 varies, depending on the traffic speed, the driver's age, experience, and his fatigue; on the average its value varies from about 0.4 to 0.7 sec. When estimating the stopping distance for designing the route in plan and profile, the driver's reaction time is usually taken as 1 sec since there may be some novices amongst the drivers in a traffic stream.

Thus the rated time lag for the total reaction of the driver is $t' = t_1 + \Delta t_1 + t_2$. At the instant when the full braking action occurs t_1 will be 1.2 sec with a hydraulic brake, and 2 sec in the case of an air brake system.

Neglecting the air resistance, the distance covered during the period of full braking action may be determined according to the formula of a uniformly decelerating movement

$$v = \sqrt{2aS_b} \tag{39}$$

where v = speed at the start of braking action, m/sec
S_b = stopping distance, m
a = absolute value of the negative acceleration during the application of brakes. Substituting $P_w = 0$ and $a = j$ in Eq. (35), we get

$$a = (\gamma_b + f \pm i)\,g \tag{40}$$

Substituting the values of a and γ_b into Eq. (39), the expression for the stopping distance becomes

$$S_b = \frac{v^2}{2g\left[\frac{\frac{M_b}{r_k}\lambda + \varphi G(1-\lambda)}{G} + f \pm i\right]} \tag{41}$$

Because of the considerable effort required to stop a vehicle in an emergency the drivers of trucks usually apply their brakes less sharply, but over a longer distance.

The factor λ is related to the braking effort P_b and to the weight of the vehicle G by the following equation

$$\lambda = \left(1 - 0.1\frac{P_b}{G}\right) \tag{42}$$

In calculations involving the determination of vertical and horizontal geometrical road elements, emergency braking is considered with the complete locking of wheels. This corresponds to the value of the factor λ equal to 0. However, in actual conditions of operation, owing to wear and dirt on the motor vehicle brakes, inaccuracy of their adjustment and lack of uniformity in distribution of effort between the wheels in the process of braking, the full theoretical value of the coefficient of adhesion is seldom realized.

This factor is taken into consideration by introducing into the formula of the stopping distance a "coefficient of operational braking performance" K_{op}. After this, the computed stopping distance becomes

$$S = \frac{v^2 K_{op}}{2g(\varphi \pm i + f)} + vt' \tag{43}$$

where v is the speed in m/sec.

According to practical measurements on vehicles equipped with hydraulic or pneumatic brakes $K_{op} = 1.4$. However, in view of the continual improvements in vehicle design and operation it is usual to assume $K_{op} = 1.2$ when determining geometrical standards for roads.

15. Standardization of Maximum Gradients on Highways

The described methods of estimating the gradients negotiable by motor cars give the means of determining the maximum upgrade for a given type of vehicle under specified conditions of loading and engine wear. Nevertheless, when performing the technical calculations for road design and the standardization of maximum longitudinal gradients one has to take into account a variety of technical and economic considerations.

The chart of dynamic characteristics gives the maximum tractive effort which a motor vehicle can develop at maximum power, i.e., at full throttle in the case of carburettor-controlled engines and maximum fuel-pump delivery in fuel-injection engines. In reality, however, engines are not called upon to operate at maximum power throughout their journeys. Moveover, the condition and degree of wear of each vehicle in any traffic stream are different. Thus the actual horsepower developed by a vehicle engine which is in need of overhaul can be 20% below that of a new engine recently run in, owing to carbon deposits in the combustion chambers, deposits on the walls of the inlet pipes, wear of parts and poor adjustments. Traffic streams comprise a variety of types of vehicles carrying various loadings. Because of this, the standardization of specifications for longitudinal gradients, based on the dynamic characteristics of one specific type of motor car, can be feasible only in a limited number of cases, mainly for industrial transport, for quarries or for building sites.

When determining the specifications for general-purpose motor roads, one must attempt to reduce to a minimum the total cost of these roads to the national exchequer, by assessing the effect of the gradients on the cost of road building and on the automobile transportation characteristics: traffic speed, fuel consumption and load-carrying capacity. At the same time it should be noted that stretches with steep longitudinal gradients form a comparatively small percentage of the total extent of the road network.

If, for instance, steep gradients were to be accepted on a road in undulating country, thereby reducing the extent of earthworks required and hence the constructional cost of the road, vehicles would be compelled to proceed on these sections in low gear. On the other hand, if traffic is to be permitted to traverse gradients at high speeds, the roads will have to be built with gentle longitudinal gradients at a considerably enhanced cost. Therefore, in determining the steepness of gradients the capital cost of the road's construction must be weighed against the long-term effects of vehicle operational costs along this road. Thus the establishment of longitudinal gradient standards becomes a technical and economic problem.

In the U.S.S.R., according to the Building Standards and Regulations, the maximum longitudinal gradients are taken in relation to the design speed, as follows:

Design speed, km/hr	150	120	100	80	60	50	40	30
Gradient, per cent	3	4	5	6	7	8	9	10

In all cases it will be good policy to design highways with longitudinal grades not exceeding 3 per cent, unless this leads to a considerable increase in the quantities and cost of the construction work.

The longitudinal gradient standards in force in the U.S.S.R. differ little from those accepted in other countries. Thus, for instance, in the Federal Republic of Germany the following maximum gradients are in use for expressways: in flat country 4%, in hilly country 5%, in mountainous regions 6% and in highlands 6.5%. On roads which are in the care of separate authorities and on rural roads the maximum permissible gradients are from 3 to 7%, depending on the country layout.

The UNO Committee in its 1950 specifications for international motorways recommended longitudinal gradients of 5% for roads in a flat country with a maximum of 6% for separate short stretches, and for roads in mountainous regions, respectively, 8 and 10%.

16. Characteristics of Combination Vehicles

One of the ways of reducing the cost of transportation is by means of articulated lorry and trailer units, generally referred to as *combination vehicles*. The use of combination vehicles, consisting of a lorry and two standard trailers, enables the load-carrying capacity of the vehicle to be increased from 2.5 to 3 times.

Analysis of vehicle service records proves that the productivity of a combination vehicle increases by 40 to 60% as compared to a single lorry, and at the same time the specific fuel consumption drops by 20 to 35%, thus reducing the operating cost by 20 to 30%.

The roads on which combination vehicles run have to comply with more stringent requirements than roads for single vehicles. This is because the motive power remains the same as for a single lorry, although the weight and the air and rolling resistances to motion are greater.

The equation of the dynamic factor becomes for a combination vehicle as follows:

$$\frac{P_p - P_{w_{cv}}}{G_l + G_{tr}} = f_1 \pm i \pm j \qquad (44)$$

where $P_{w_{cv}}$ = air resistance to the movement of the combination vehicle
G_l = weight of the lorry
G_{tr} = weight of the trailers
f_1 = rolling resistance factor for the vehicle.

The dynamic factor of a combination vehicle is less than that of a single one, but the resistance of the vehicle is higher. Because of the friction in the articulation couplings and in the slewing circle, and also owing to the pitching of the trailers in motion and to the impacts due to jolts, the rolling resistance of a combination vehicle increases with the number of trailers attached. At the same time

the air resistance increases because for each additional trailer there is additional air friction on its sides with the formation of air eddies behind it.

The values of combination vehicle rolling resistance on asphalt concrete pavement and of air resistance are given in Table 4.

TABLE 4

Composition of combination vehicle	Relative rolling resistance	Relative air resistance
Single lorry	1.00	1.00
Ditto with one trailer	1.08	1.32
Ditto with two trailers	1.10	1.59
Ditto with three trailers	1.12	1.84

Additional resistance occurs when starting a combination vehicle, and this should not be overlooked. To allow for this, the rolling resistance factor should be increased 1.5-2.5 times for summer conditions, and 2.5-5 times for winter conditions.

Still less favourable results are obtained for adhesion. The equation of combination vehicle propulsion is

$$\frac{\varphi G_{ad} - P_{w_{cv}}}{G_l + G_{tr}} = f_1 \pm i \pm j \tag{45}$$

Since the combination vehicles have relatively small speeds, the air resistance $P_{w_{cv}}$ can be neglected in Eq. (45).

To increase the safety of traffic, modern trailers are fitted with servo-brakes controlled from the lorry. Without servo-assistance the braking of the unit presents appreciable difficulties, especially on gradients.

$$\frac{-\varphi P_b - P_{w_{cv}}}{G_l + G_{tr}} = f_1 \pm i \pm j \tag{46}$$

In the above expression P_b is the load on the brake axles.

The distance covered by a combination vehicle during the braking period is appreciably longer than that for a single vehicle. Moreover, sharp application of brakes creates the danger of the trailers skidding, jack-knifing or running into the lorry.

Consequently, when designing a road for the accommodation of combination vehicles, the maximum longitudinal gradients should

be reduced. According to some authorities, if a road is to be used by combination vehicles, the maximum gradients should be limited to 4% irrespective of the category of the road.

17. Fuel Consumption and Tyre Wear in Relation to Road Conditions

A substantial part of the cost of transportation is attributable to the cost of fuel and lubricating oil required by the vehicle, the amount of which depends on the road and traffic operating conditions.

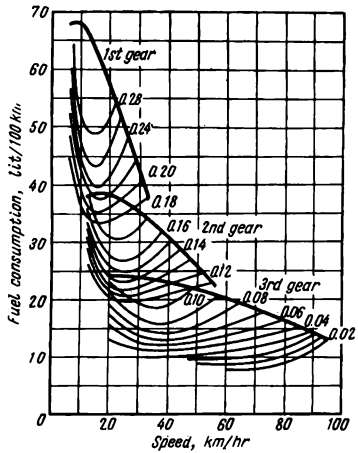


Fig. 29. Diagram of economic characteristics

For estimating fuel consumption a diagram of the vehicle economic characteristic is used, giving the curve of fuel consumption in litres per 100 km for a variety of road conditions and for varying speeds (Fig. 29). Since time-speed-distance calculations are based on the condition that the throttle is fully open, the same conditions are accepted for plotting the economic characteristic. For the performance in each gear the economic characteristic is plotted as a series of curves, each of which relates to a definite value of the total road resistance, i.e., resistance to motion and the additional resistance when travelling up a gradient.

The automobile economic characteristics can be calculated or obtained experimentally.

For a vehicle travelling with a speed V km/hr the engine has to develop a brake horsepower equal to

$$N_{eng} = \frac{V \sum P}{3.6\eta} \tag{47}$$

where $\Sigma P = (P_w + P_f + P_i + P_j)$ = sum of resistances encountered by the vehicle in motion (see Sec. 10)

η = efficiency of the vehicle transmission system.

Upon substituting the values of the resistances, we obtain for a uniform speed condition

$$N_{eng} = \left(\frac{K\omega V^2}{3.6^2} + G\psi\right)\frac{V}{270\eta} \text{ hp} \tag{48}$$

where $\psi = f + i$ is the factor of road resistances.

Also, the specific fuel consumption per 1 hp in grams per hour is given by

$$q_b = \frac{632}{H_l \eta_{eng}} \tag{49}$$

where 632 = number of calories equivalent to a work of 1 hp done during one hour

H_l = lower calorific value of the fuel, kcal/kg

η_{eng} = effective efficiency of the engine.

From this we obtain the fuel consumption

$$Q_s = \frac{q_b N_{eng}}{1{,}000\gamma} \text{ lit/hr} \tag{50}$$

where γ is the specific weight of the fuel.

When plotting the diagram of economic characteristics it is customary to express fuel consumption in terms of litres per 100 km of travel. In this case, taking into consideration expressions (50) and (48), the fuel consumption will be

$$Q_{100} = Q_s \frac{100}{V} = \frac{0.233\left(\frac{K\omega V^2}{3.6^2} + G\psi\right)}{\gamma\eta} \text{ lit/100 km} \tag{51}$$

Figure 30 shows the economic characteristics of Soviet automobiles in direct drive on smooth horizontal stretches of road with good surfacings.

The two diagrams — vehicle economic and dynamic characteristics — provide a method of finding the running time for various road stretches having diverse profiles and types of surfacing, and of determining the overall fuel consumption.

The curves of economic characteristics show points of minimum fuel consumption which correspond to speeds that are often called economic speeds. This appellation is incorrect, since the economic use of motor transport is determined from the transportation work it performs. In the majority of cases, when speeds exceed the so-called economic speeds, the extra consumption of fuel may be more than

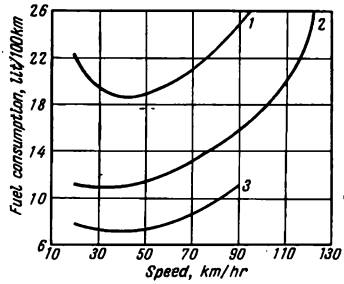


Fig. 30. Diagrams of economic characteristics of Soviet motor cars, using direct drive, moving along high-quality road surfacings:
1—ZIL-110; 2—GAZ-12; 3—Moskvich-402

compensated by a reduction in the cost of transportation, since quicker turn-around permits a greater rate of handling passengers and goods.

The fuel consumption of a vehicle moving on a given road stretch can be determined by diagrams of the dynamic and economic characteristics. At first the speeds of travel along separate stretches must be determined. Corrections are then made to take account of the actual traffic conditions: speed limits, movement down a gradient while braking with the engine, etc. This allows the fuel consumption to be determined for each stretch according to speeds and the road resistances $\psi = f + i$. This estimate can be accomplished conveniently by a graphical method, as shown in Fig. 31.

By analyzing the road profile, one determines the length of the stretches from l_1 to l_n with equal values of road resistances, which can be travelled in one or other gear, and locates on the chart of the dynamic characteristic the corresponding traffic speeds. If below the speed coordinate of the dynamic characteristic the economic characteristics are traced, then with the help of the latter it is possible to find the corresponding fuel consumption Q in lit/100 km.

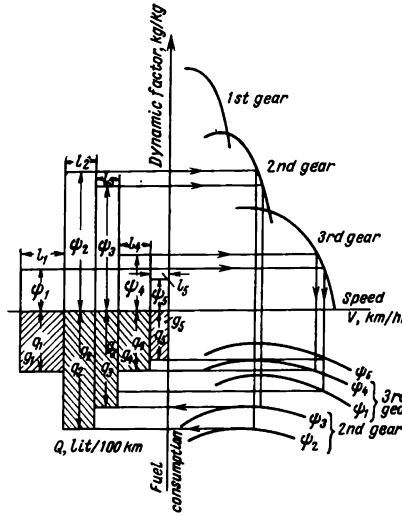


Fig. 31. Graphical method of determining fuel consumption

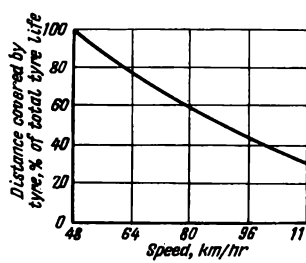


Fig. 32. Relation between tyre wear and vehicle speed

This enables the consumption for each stretch $q = Ql/100$ litres to be determined. Corrections should be introduced in the calculation to take account of the actual traffic conditions, i.e., speed limitations, bypasses, stops at cross-roads, descents while braking with the engine, etc.

The tyre wear also depends on the speed of the vehicle on the various road stretches—high speeds increase tyre wear. This is caused by an appreciable heating of tyres, also by an increase of the magnitude of impacts against irregularities of the road pavement (Fig. 32). The wear of the tyres also grows on curvilinear road stretches (see Sec. 21).

CHAPTER 4

REQUIREMENTS FOR HORIZONTAL ROAD ELEMENTS

18. Traffic Capacity and the Required Number of Lanes

The carriageway of the road must be of a width sufficient to secure the safe passage of vehicles travelling at the design speed and moving either in a single stream, or in several streams depending on the intensity of traffic.

The strip of carriageway conditionally occupied by a vehicle travelling along the road is termed a *traffic lane*. Its width comprises the overall width of the vehicle and side strips necessary for safe steering. The maximum number of vehicles which can pass along a traffic lane in a unit of time is called the *lane capacity*.

The latter depends on the traffic speed and the state of the surfacing which is defined by its coefficient of adhesion.

There are several theoretical methods for calculating the capacity of a traffic lane, which are based on the consideration of a single stream of vehicles following each other, ruling out the possibility of overtaking. The speed of all the vehicles is taken as constant. The distance between vehicles in a traffic stream is a function of their speed. This distance must be sufficient to permit application of the brakes and bringing the vehicle to a standstill without end-on collision in case of an emergency stop.

There is a time lag between any vehicle commencing some control action—by braking sharply—and the following vehicle taking similar action. This is termed the reaction time t^1; the distance travelled by the following vehicle in this time being $l_1 = t^1 v$.

It is usual to assume a reaction time of 1 second, from which $l_1 = v$.

Since the braking efficiencies of the two vehicles may differ, the stopping distance of the first may be shorter, and then the second vehicle will approach it and reduce the distance between them by

$$l_2 = L_r - L_f = \frac{v^2 (K_r - K_f)}{2g (\varphi \pm i + f)} \tag{52}$$

where L_r and L_f = stopping distances of the rear or following and the forward vehicles, respectively

K_r and K_f = coefficients of operational braking efficiency of each vehicle.

For the safety of traffic it is necessary to provide for a certain safety distance between the vehicles coming to a standstill, l_3, of an order of 5 to 10 m.

Hence the safety distance between two moving vehicles following each other is

$$s = v + \frac{v^2 (K_r - K_f)}{2g (\varphi \pm i + f)} + l_3 \text{ m} \tag{53}$$

Each vehicle in a traffic stream effectively occupies a length of road $s + l_4$ called the *headway*, where l_4 is the length of the vehicle and s the spacing between successive vehicles. The time required by a vehicle to cover the headway is therefore $t = \frac{(s + l_4)}{v}$ sec.

The number of vehicles which can pass during an hour is

$$N = \frac{3{,}600}{t} = \frac{3{,}600v}{v + \frac{v^2 (K_r - K_f)}{2g (\varphi \pm i + f)} + l_3 + l_4} \tag{54}$$

where v is the speed in m/sec.

For assessing lane capacity various assumptions are made in respect to the braking conditions and brake efficiency of the forward and following vehicles. The most usual is the assumption of instantaneous stopping of the forward vehicle ($K_f = 0$). This could be visualized as an object falling off a lorry, which presents a danger for the vehicle following behind. In this case

$$N_1 = \frac{1{,}000V}{\frac{V}{3.6} + \frac{K_r V^2}{254 (\varphi \pm i + f)} + l_3 + l_4} \tag{55}$$

where V is the speed in km/hr.

Mathematical investigation of Eq. (55) shows a maximum of an order of 1,100 to 1,600 veh/hr, which corresponds to traffic speeds between 20 and 40 km/hr. With a further increase in speed the lane capacity steadily decreases (Fig. 33).

The second assumption is that the state and operational conditions of the brakes of both vehicles are identical ($K_f = K_r$). In this case

$$N_2 = \frac{1{,}000V}{\frac{V}{3.6} + l_3 + l_4} \tag{56}$$

where V is the speed in km/hr.

This assumption may be realized only in the comparatively infrequent case of a convoy composed entirely of vehicles of the same type, when the lane capacity increases with the speed.

The numerous investigations of the traffic operation conditions carried out in various countries lead to a series of empirical formulas

having the same structure as expressions (55) and (56), but in which in accordance with local conditions different coefficients are used for V and V^2, and different numerical values for l_3 and l_4.

In spite of this coincidence, the theoretical calculations of the traffic capacity of a single lane have a very limited significance, since in reality vehicles in a traffic stream move with varying speeds. Furthermore, it is usual for vehicles to overtake slower ones, which is not allowed for in the above formulas. On a carriageway having

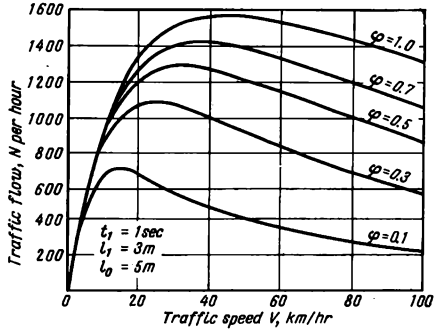


Fig. 33. Capacity of a traffic lane for different values of the coefficient of adhesion

two traffic lanes, the overtaking vehicles draw out into the opposing traffic lane and, beginning from a certain intensity, cause a general slowing down of traffic. Investigations carried out in the U.S.S.R. and in Great Britain have shown that the speed of traffic streams decreases proportionately to the traffic intensity (Fig. 34). For this reason it is important to use the results obtained during the hours of maximum traffic intensity for the assessment of road traffic capacity in the conditions of mixed traffic.

Practically, the capacity of a single lane with a dry surfacing is about 1,000 veh/hr, and with a slippery surface approximates 500 veh/hr.

When building new roads for a traffic intensity not exceeding 5,000 veh/day, the carriageway may comprise two traffic lanes, i.e., the maximum traffic intensity for one lane is assumed to be 250 veh/hr. For an average daily flow of 5,000 to 10,000 vehicles per day, the road should be designed to allow for four traffic lanes, while for still heavier flows, the number of lanes must be determined by calculation.

When travelling up steep inclines, laden trucks are compelled to change down to low gear, and this causes an excessive slowing down of the passenger car traffic. In order to avoid the consequent lowering of traffic capacity, an additional traffic lane is often provided for the up-grade traffic in Western Germany and in the U.S.A., specifically to accommodate heavy trucks and combination vehicles.

Three-lane carriageways are not built in the U.S.S.R., as it is considered that there would be insufficient utilization of the central lane, while there is a hazard of head-on collision between

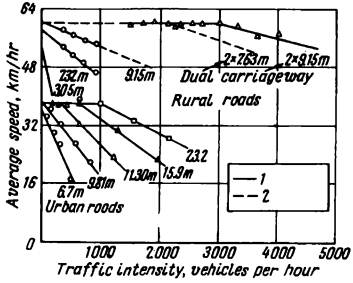


Fig. 34. Mean traffic speed on roads versus flow and lane width:
1—experimental values; *2*—assumed values

vehicles when overtaking occurs in each direction. However, experience with three-lane carriageways shows that with suitable discipline of drivers, when vehicles draw out into the central lane only in cases of strict necessity, the capacity of a three-lane carriageway is much greater than that of a two-lane one.

19. Width of Carriageways and Shoulders

When a large number of vehicles traverse a single traffic lane, their wheel tracks do not coincide exactly, but lie within a strip 50 to 60 cm wide, termed the *tread path.*

The higher the average speed of the traffic, the wider will be the traffic lane required.

The width of the nearside traffic lane (Fig. 35) can be determined by the following formula:

$$B = \frac{b+c}{2} + x + y \tag{57}$$

where b = width of the vehicle body
c = automobile wheel track (distance between the wheel centre lines)
x = distance from the vehicle body to the adjacent traffic lane
y = distance from the wheel track centre line to the edge of the carriageway.

If the edge of the carriageway is defined by a raised kerb, the distances y and x—determining the width of the traffic lane—are measured from the edge of the vehicle body.

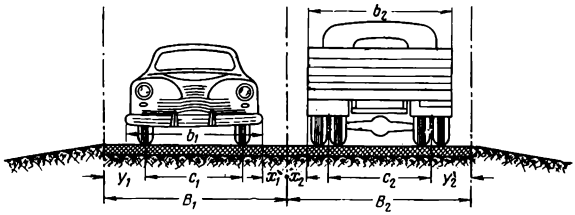


Fig. 35. Sketch for determining the width of a traffic lane

If the road has several traffic lanes, the width of the centre lane—not next to the edge of the carriageway—is equal to

$$B = b + x_1 + x_2 \tag{58}$$

where x_1 and x_2 are the distances from the vehicle body to the adjacent traffic lanes.

The values of x and y depend on the vehicle traffic speeds and are standardized according to the investigations of traffic and its safety conditions.

On the basis of measurements determined by the filming of actual distances between vehicles when crossing and overtaking each other, M. S. Zamakhayev proposed the following relation between the distance x or y and the speed:

in the case of opposing traffic

$$x = y = 0.5 + 0.005V \tag{59}$$

and in the case of traffic in the same direction

$$x_1 = x_2 = 0.35 + 0.005V \tag{60}$$

where the speed of traffic V is expressed in km/hr, and the distances x and y in m.

When determining the design width of a traffic lane two separate cases should be investigated. For this end we must consider:

(1) passenger cars, which have narrow bodies, but travel at high speeds;

(2) trucks, etc., which have wide bodies, but travel at lower speeds.

Determination of the carriageway width is both a technical and an economic problem. On roads which are anticipated to carry only modest flows, one may accept a lesser width of traffic lane in order to keep down the capital cost of construction, it being understood that this will entail a reduction in speed on the infrequent occasions when overtaking is necessary.

Carriageway width is also related to driver discipline. Many drivers persist in driving too far out from the edge of the carriageway, and the unjustified increase of the distance y leads to the lowering of the traffic speed and increases the danger of road accidents. One of the measures for counteracting this undisciplined action is the painting of white lines marking the lanes on the road, or their indication by means of reflecting studs.

The BS and R provide for the following lane widths, which in general are quite satisfactory for the average modern conditions of traffic over roads:

On roads of classes I and II	3.75 m
On roads of class III	3.5 m
On roads of class IV	3 m
On roads of class V the carriageway may have a width of	4.5 m

On roads of classes I and II, and on roads of class III in especially unfavourable soil, hydrologic and climatic conditions, edge strips, each 0.75 m wide, are constructed on the shoulders next to the improved pavement. These strips are made from concrete slabs, or from crushed stone and gravel materials processed with various binders as well as from separate stones. An elevated curb may be installed.

The edge strips must differ in colour from the main pavement to ensure good vision of the latter at night.

The widths of the traffic lanes in other countries are similar to those used in the U.S.S.R. The traffic lane width of 3.5 m is recommended by the UNO Committee for the International Highway Network. In the U.S.A., on heavily used roads, the lanes are made 3.65 m wide, and in the F.R.G. they are 3.75 m with complete lateral separation of opposing traffic streams on autostrades and with two lane traffic of heavy freight vehicles on local roads.

In rolling country, motor roads consist of alternating upgrades and downgrades. The vehicles travelling down a gradient gain an

impetus, and on the lower part of the slope travel with greater speeds than on its upper part.

Taking into account the relation established above between the traffic lane width and the traffic speed, the altering of the carriageway width according to speed is justified in conditions of variable-speed traffic.

This is taken into consideration by the recommendation of the BS and R consisting in that in the lower part of concave vertical curves with an algebraical difference between adjoining grades of over 6 per cent, each lane of classes III and IV roads must be made 0.25 m wider over a distance of at least 100 m.

The construction of shoulders and verges on both sides of the carriageway provides firm edges on which vehicles may pull up. During the renewal and repair of surfacing the materials and implements may be stored on the shoulders and verges. It is desirable that the width of the shoulders be such that the stationary vehicles do not obtrude into the carriageway. For the majority of types of vehicles a width of 2.5 to 3.0 m is sufficient.

On high-class roads and within populated areas the shoulders are stabilized with gravel, by paving, or by treatment with binding agents. Without this the road would be soiled during the wet season by the mud carried over onto the carriageway on the wheels of the vehicles.

A cheaper method of stabilization is the building of a wide pavement, forming a hard shoulder. The boundary of the carriageway is marked by means of a coloured line.

20. Problems of Traffic Motion on a Curve

A vehicle moving with a constant speed v m/sec along a circular curve of radius R is subject to the action of a centrifugal force

$$C = \frac{mv^2}{R} \tag{61}$$

The centrifugal force, applied normal to the direction of vehicle movement, has an appreciable influence on the stability and subsequent motion of the vehicle. Apart from the overturning effect and the sideways thrust, the centrifugal force leads to considerable alteration in the distribution of loading between the nearside and offside wheels. In this case it is more difficult for the driver to hold the vehicle within the limits of the traffic lane.

The effect of the centrifugal force on the vehicle causes a transverse deformation of the tyres, increases their wear, and also increases fuel consumption. At night time the conditions of vehicle

movement on a curve are complicated by the head lamps illuminating the road in front of the vehicle to a lesser distance than is required by the safe sight distance ratings.

The influence of these adverse factors increases as the radius of the curve in plan decreases. Therefore, in the interests of safety, comfort and the economy of vehicle operation at design speed the bends must be set out to a suitable minimum curvature.

In general, the expression for determining the curve radius in plan can be obtained from the following considerations.

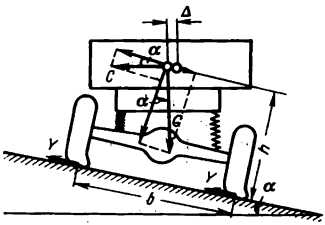


Fig. 36. Forces acting on a vehicle travelling along a curve

When in motion along a curve, the vehicle is subject to two transverse forces, applied at its centre of gravity (Fig. 36):

(a) the centrifugal force C, directed horizontally towards the outside of the curve [see Eq. (61)];

(b) the component of the vehicle weight, parallel to the road transverse gradient and equal to mgi; depending on the direction of the transverse gradient this component may have a positive or a negative value.

The projection of both these forces on the direction of the transverse gradient leads to the following equation:

$$Y = \frac{mv^2}{R} \cos \alpha \pm mgi \tag{62}$$

in which Y is the total force tending to slide the vehicle off the road, and called the *lateral force*.

Since the angle α is small ($\cos \alpha \approx 1$), its effect can be neglected, whence

$$Y \approx \frac{mv^2}{R} \pm mgi \tag{63}$$

From which one may derive

$$R = \frac{v^2}{g\left(\frac{Y}{mg} \pm i\right)} \tag{64}$$

This equation shows that the required radius of curvature depends not on the absolute value of the lateral friction force Y, but on its ratio to the weight of the vehicle: $\mu = Y/mg$. This ratio is called the *coefficient of lateral friction force* or *lateral friction.*

Substituting μ for Y/mg in Eq. (64) we get

$$R = \frac{v^2}{g(\mu \pm i)} \tag{65}$$

For practical application of this expression it is necessary to specify the tolerated value of the coefficient of lateral friction force μ.

21. The Coefficient of Lateral Force

The design value of the coefficient of lateral force must always be selected with the aim of ensuring the stability of the vehicle, the convenience of its steering and the nature of usage, as well as the economical operation of the vehicle on curvilinear road stretches.

The resistance of a vehicle to overturning is ensured only when the value of the restoring moment is greater than that of the overturning one (see Fig. 36).

Taking moments about the centres of the outside wheel contact areas, we obtain

$$Yh = mg\left(\frac{b}{2} - \Delta\right)$$

whence

$$\mu = \frac{Y}{mg} = \frac{b - 2\Delta}{2h} \tag{66}$$

In this equation account is taken of the fact that owing to spring deformation and tyre elasticity there will be a lateral shift of the centre of gravity of the vehicle over a distance of Δ.

To determine the coefficient of lateral friction force it is necessary to examine the vehicle dimensions, in particular, the ratio of its track width b to the height h of its centre of gravity.

During an experiment on passenger cars in which the developed value of Y was 700 kg, the value of the lateral shift Δ was found to be approximately $0.2b$. The ratio b/h for modern passenger cars varies from 1.8 to 2.5 whilst for trucks it varies from 2 to 3 and for

coaches from 1.7 to 2.2. Assuming for calculation purposes the minimum value of b to be $1.7h$, it will be seen that for the vehicle to resist overturning it is necessary that the coefficient of lateral friction force be not greater than 0.6.

In normal conditions of vehicle operation and at usual traffic speeds the coefficient of the lateral force does not attain this value.

Resistance to skidding. During the movement of a vehicle along a curve the tyre adhesion to the surfacing prevents the vehicle from side-slipping under the action of the centrifugal force.

The lateral force Y and the tractive or braking effort P, applied to the driving wheel of the vehicle, create in the area of the wheel contact with the surfacing a total displacing force Q directed at an angle to the line of motion (Fig. 37). For vehicle stability it is necessary that the following condition be satisfied:

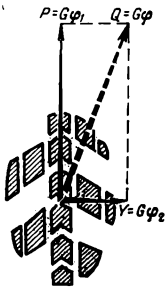


Fig. 37. Ratio between the lateral and longitudinal forces acting on a vehicle wheel

$$\sqrt{Y^2+P^2}=Q\leqslant G'\varphi \qquad (67)$$

where G' = pressure of the driving wheel on the surfacing

φ = coefficient of adhesion between the tyre and the surfacing.

It is assumed in this case that the value of Q does not depend on the angle which its line of action forms with the direction of vehicle motion.

Under the action of the centrifugal force the distribution of loading between the wheels is modified. When the lateral force is considerable the tractive effort on the lightly loaded wheel may become greater than the force of adhesion, which will result in spinning of the wheel. The coefficient of adhesion in this case will decrease still further, and the vehicle may skid.

Analysing the conditions of vehicle resistance to sideways slip, it is necessary to take into account the developed coefficients of adhesion in both the longitudinal and transverse directions which transmit the tractive or braking efforts and create the resistance to skidding (Fig. 37). These are interrelated as follows:

$$\varphi_2=\sqrt{\varphi^2-\varphi_1^2} \qquad (68)$$

To ensure stability on a curve it is necessary that the coefficient of lateral friction force μ should not be greater than the coefficient of lateral adhesion φ_2. Otherwise the vehicle will be displaced from its selected path. Meanwhile, the greater the component of the total

coefficient of adhesion used in overcoming the longitudinal gradient or in braking, the lower will be the value of the coefficient of lateral adhesion available for resisting the sliding movement of the vehicle.

Therefore, when determining suitable minimum horizontal radii one has to take account of the relationship between the coefficients φ_1 and φ_2. The greater the permissible value of φ_1, the lesser will be the value of μ at which the probability of skidding will occur.

Recommended values of φ_1 and φ_2 are

$$\varphi_1 = 0.8\varphi \text{ to } 0.7\varphi$$

From which

$$\mu = \varphi_2 = 0.6\varphi \text{ to } 0.7\varphi$$

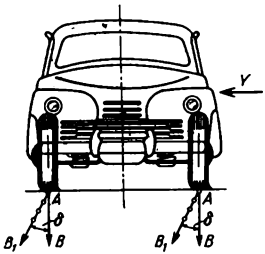


Fig. 38. Yaw of vehicle wheels (AB is the initial direction of vehicle propulsion; AB_1 is the direction of motion due to slip angle)

Provision for the comfort of road vehicle passengers when travelling around a curve. The centrifugal force arising when an automobile enters a curve affects the passenger, who feels a shock or impulse which tends to fling him sideways. Therefore, it is important that the magnitude of the centrifugal force should not exceed a limiting value at which the passenger feels discomfort when travelling along a curve.

Tests show that with a lateral friction force coefficient of $\mu = 0.1$ a passenger not looking at the road cannot tell whether the vehicle is running along a straight or a curve. With $\mu = 0.15$, the curve is felt slightly, and at $\mu = 0.2$ the passenger feels the motion clearly and suffers some slight discomfort. With $\mu = 0.3$ the transfer from a straight to a curve is sensed as a shock which pushes the passenger sideways. Therefore, to ensure the passenger's comfort when travelling on a road, the value of the coefficient of lateral force μ on curves should be limited to a maximum of 0.15 to 0.20.

Economy of motor traffic. The lateral force acting on the wheel causes yawing, i.e., partial side slipping or crabwise motion causing it to roll skew to its wheel plane and involving extensive tyre deformation (Fig. 38).

Experimental investigations show that, with a slip angle $\delta < 4\text{-}5°$ this slip angle is directly proportional to the side force acting at right angles to the wheel plane of rotation

$$\delta = kY \tag{69}$$

where Y = lateral force, kg
k = coefficient of proportionality, depending on the resilience of the pneumatic tyre in the lateral direction.

For the various types of tyres for modern passenger cars, which were subject to test, the values of k were between 1,700 and 3,990 kg/rad. Y. A. Dolmatovsky has found that the value of

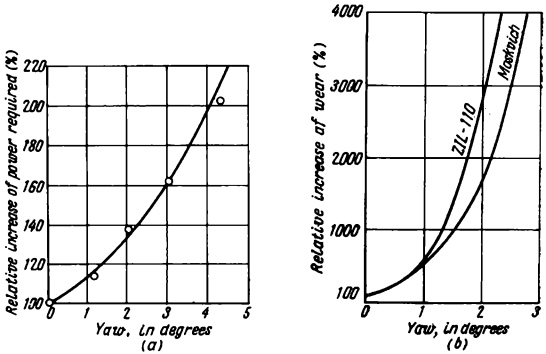


Fig. 39. Power required (*a*), and tyre wear (*b*) due to yaw of a rolling wheel, according to G. A. Gasparyantz

k for tyres manufactured in the U.S.S.R. depends on the size of the tyres, and this relation can be expressed by the formula

$$k = 5b\,(D + 2b)\,(p + 1) \text{ kg/rad} \tag{70}$$

where b = cross-section width, inches
D = diameter of the rim, inches
p = tyre air pressure, kg/cm².

Experimental investigations show that the increase of the slip angle results in a sharp rise in engine horsepower consumption for wheel rotation and in tyre wear (Fig. 39). The limitation of the lateral force to a value at which the slip angle would not exceed 1°, would still occasion a fivefold increase in tyre wear. The additional engine horsepower required, which calls forth a relevant increase of fuel consumption, may be as high as 15%.

The coefficient of lateral force, corresponding to these conditions, is

$$\mu = \frac{Y}{G} = \frac{\delta}{kG} \approx 0.1$$

Thus, in order to make automobile transportation economical curves should be set out to such radii that the coefficient of the lateral force does not exceed the value $\mu = 0.1$.

The yaw effect should be given careful consideration in road layout theory. The wheel yaw is also the result of the action of a wind blowing across the road. The driver compensates this effect by inclining the axis of the wheels to his course by turning the steering wheel. The motion in this case is accomplished on laterally deformed tyres. If the lateral force ceases instantaneously—for example, when entering a calm zone—the deformed tyres redress immediately, and owing to different sideslip factors of the forward and rear wheels a tendency to skid occurs. Because of this phenomenon, in the F.R.G. for instance, on motorways in localities subject to the persistent action of side winds from the sea, special signs are displayed warning of the danger of skidding due to the action of the wind.

22. Selection of Radii for Horizontal Curves

Minimum radii for horizontal curves on motor highways have to be selected in accordance with one of the following two cases:

(a) minimum radii which ensure traffic safety in difficult topographic conditions or in a densely populated country, where an increase of radius will lead either to a substantial increase in the volume of earthworks or to the necessity of demolishing valuable buildings;

(b) when the radius is being chosen for a layout in an open country with no obstructions of any kind limiting its magnitude.

In the first case one has to aim at the safe progress of traffic and at reducing the cost of transport at the design speed in favourable road conditions and within the maximum permissible value of the coefficient of the lateral friction force.

In the second case, the design speed is to be maintained on wet surfacing with values of μ corresponding to a comfortable journey in a motor vehicle.

Analysis of the movement of vehicles along curves makes it possible to compare the maximum permissible values of the coefficient μ for various specified requirements concerning vehicle stability and utilization (Table 5).

Since driving at high speeds on ice-covered roads is impossible, seeing that the slightest maladjustment of brakes and the road camber effect could be the cause of skidding even if the brakes are balanced and applied on the straight, all calculations should assume a wetted state of road surfacing. With exceptionally unfavourable conditions for route layout, that require the use of minimum curve radii, the calculation could be based on a value of $\mu = 0.2$, which

TABLE 5

Requirements to be satisfied	Maximum permissible values of the coefficient of lateral friction force μ in relation to the state of the road surfacing		
	Dry (φ=0.6)	Wet (φ=0.3)	Ice-covered (φ=0.2)
Resistance to overturning	0.60	0.60	0.60
Resistance to skidding	0.36	0.20	0.12
Provision of comfort for passengers' travel	0.15	0.15	0.15
Economy of vehicle operation	<0.10	<0.10	<0.10

guarantees resistance of the vehicle to skidding, but lowers the comfort and economy of road use along this curve.

With relatively favourable topographic conditions it is reasonable to base calculations of minimum radius magnitude on $\mu = 0.1$. Thus, for example, in the U.S.A. $\mu = 0.16$ is permissible for a speed $V = 48$ km/hr and $\mu = 0.12$ for a speed $V = 112$ km/hr.

In the Soviet Union the following minimum curve radii for horizontal curves are specified:

Road class	I	II	III	IV	V
Minimum radius of curves in ordinary conditions, m	1,000	600	400	250	125
Ditto, on difficult sections of broken terrain, m	600	400	250	125	60
Ditto, on difficult sections of mountainous country, m	250	125	100	60	30

To increase traffic safety on curves when it is impossible to increase their radii a steep straight crossfall can be provided pitching towards the centre of the curve; this is termed *super-elevation.* The magnitude of this crossfall can be greater than the camber on straight stretches (see Sec. 23).

If the road is built in an open flat country, the increase of the radius reduces the length of the road, as well as the constructional and transport costs. Therefore, in favourable conditions it is recommended that curves of the maximum possible radii be selected (3,000 to 5,000 m).

On curves with small radii safe conditions for travelling at the design speed at night are seldom ensured, since for traffic safety it is necessary for the road surface to be illuminated by the head lamps to a specific distance allied to the design speed of the road.

The importance of satisfying the safety requirements at night is demonstrated by the fact that although the traffic intensity is about 10 times less at night than in the daytime, half of the accidents occur at night.

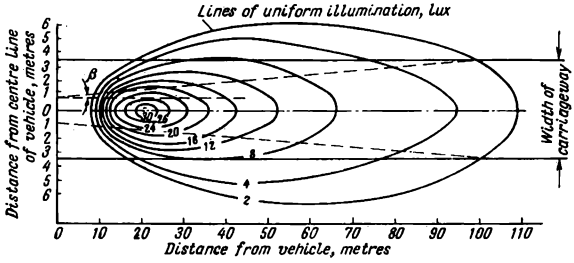


Fig. 40. Head-lamp illumination of the road

The optical arrangements of modern head lamps concentrate the light beam into an elliptical form which can be defined by the angle of light dispersion of the head lamp β (Fig. 40).

The minimum permissible illumination of the road surface is usually assumed to be 2 lux.

The modern long-distance head lamps provide good visibility, in the absence of opposing traffic, up to about 175 m, and this may reach a maximum of 250 m, which, however, is still less than the rated safe sight distance. In case of opposing traffic the dazzling effect of the on-coming vehicle head-lamp lights causes visibility to be sharply reduced—to 20-70 m.

The problem facing the industry is to increase the illumination distance and at the same time to reduce the dangers from dazzle to drivers of on-coming vehicles. The requirements for visibility with head-lamp lighting differ from those for sight distance during the daytime. It is reckoned sufficient if a driver can see the outlines of an obstacle on a road at a sufficient distance from the vehicle, thrown up as a silhouette against the road surface background.

The magnitude of the radius R, at which the road visibility on a curve would correspond to the safe sight distance S, can be deter-

mined from the following consideration (Fig. 41). The angle at the centre β, subtended by an arc of a length $(S + l)$, where l is the length of the vehicle, is given (in degrees) by

$$\beta = \frac{180\,(S+l)}{\pi R} \qquad (71)$$

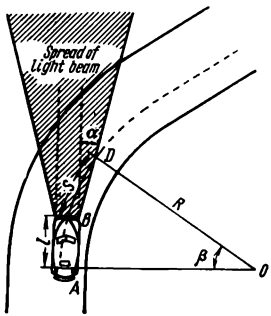


Fig. 41. Effect of head-lamp beam on determination of curve radius

From Fig. 41 we can see that $\beta = 2\alpha$. Equating α and the obtained value of β, the magnitude of the radius R is determined

$$R = \frac{28.6\,(S+l)}{\alpha} \qquad (72)$$

Neglecting the length of the vehicle l, which is much less than the safe sight distance S, we obtain the approximate expression for the curve radius

$$R \approx \frac{30S}{\alpha} \qquad (73)$$

The angle of light dispersion for modern head lamps α is approximately 2°.

For the sight distances of 100 to 300 m specified for high-class, roads, the curve radii at which the head lamps illuminate the carriageway to these distances should be from 1,500 to 4,500 m.

23. Additional Elements on Curves of Small Radius

To ensure traffic safety at the design speed on curves of small radii, a number of additional provisions have been introduced into the construction of roads, such as *super-elevation*, *extension of the width of the carriageway*, and *transition curves*. For the improvement of visibility along curves of small radii the *toes of cuttings* must be set back on the inner side of the bend, etc.

Super-elevation. When rounding a curve, particularly adverse conditions are created for vehicles moving on the outer edge of the carriageway having a normal camber. In this case the stability of the vehicle decreases sharply, and if high-speed traffic is to be maintained curves of large radius must be set out. However, local conditions do not always permit this. To increase the stability of the vehicle on curves of small radius adverse camber is eliminated by the introduction of a *straight crossfall*—a super-elevation with the gradient of the carriageway and shoulders falling towards the centre

of the curve. The amount of super-elevation is calculated to provide for the vehicle's resistance to side slipping.

The super-elevation required to maintain the traffic speed v m/sec at a given radius is determined by transforming expression (65)

$$i_{s-el}=\frac{v^2}{gR}-\mu=\frac{v^2}{gR}-\varphi_2 \tag{74}$$

where φ_2 is the coefficient of lateral adhesion between the wheel and the road, because of all the criteria examined in Sec. 21, the critical value, in this case, is the resistance of the vehicle to skidding.

The amount of super-elevation theoretically required for high-speed traffic may be considerable. Such super-elevations are applied, for instance, on motor tracks intended for automobile racing.

On modern highways the super-elevation is usually limited to about 6%. In regions where there is no prolonged snow or ice covering, an increase of the super-elevation up to 10% may be admissible. However, such steep super-elevations are not convenient for trucks travelling at a speed appreciably lower than the design speed.

In regions with frequent fogs and long periods of ice-covered pavements, as well as on mountain roads where the pavements are frequently covered with ice, the maximum super-elevation of the roadway on curves is taken not over 4 per cent, correspondingly increasing the radii of curves.

Experience of operating high-speed highways shows that super-elevations affect the drivers psychologically in that the same traffic speed is maintained on curves as on the adjacent straight stretches. Without super-elevation the speed is unintentionally reduced. For this reason, super-elevations are not regarded only as belonging to curves of very small radii, and in the U.S.S.R. they are built on all curves of radii smaller than 3,000 m on roads of class I and 2,000 m on roads of other classes. In some other countries on roads with high design speeds, super-elevations are also provided for curves of large radii. For example, on the F.R.G. highways super-elevations are provided on all curves without exception, maintaining the normal crossfall of 1.5% on curves of radius exceeding 5,000 m but increasing it up to 6% for curves of a smaller radius.

Depending on the radius of the horizontal curve, the following super-elevations are employed:

Radii of horizontal curves, m	3,000	3,000	(2,000)-1,000	1,000-700	700-650	650-600	Less than 600
Super-elevation of curve, per cent	ridged section	2-3	(2-3)	3-4 (3-4)	4-5 (4)	5-6 (4)	6 (4)

Notes: 1. The smaller values of the banks on curves correspond to the larger radii.

2. The figures in parentheses relate to regions where the pavements are frequently covered with ice.

The change of grade from straight crossfall to normal cross-section camber is achieved within a short length of road known as the *transition* (Fig. 42).

The transition from a cambered section to a uniform crossfall is achieved by swinging the line of the outer part of the carriageway

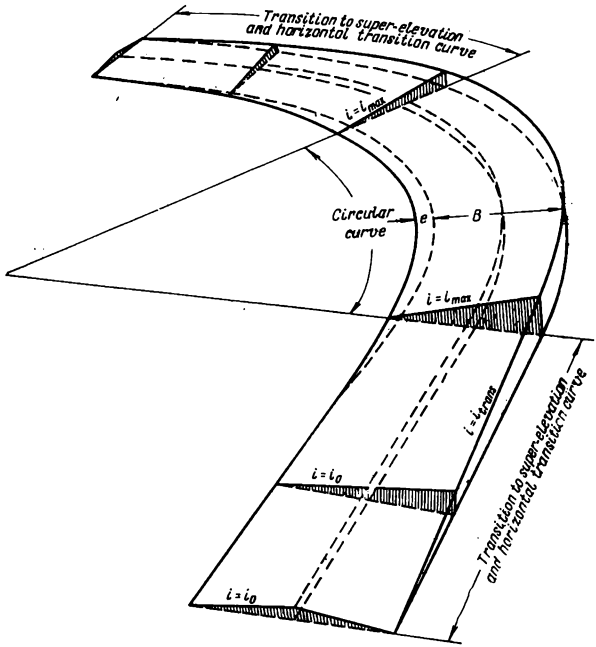


Fig. 42. Diagram showing application of super-elevation to a cambered carriageway

cross-section about the road centre line, until the cross-section becomes a straight fall and equal to the slope of the camber; thereafter the straight cross-section is rotated about the inner edge of the carriageway until the required degree of super-elevation is attained.

Figure 43 shows a layout for the super-elevation of a motorway with carriageways having a straight crossfall. On a carriageway which falls towards the centre of the curve, the super-elevation is applied by increasing the transverse gradient. On the outer carriageway the adverse crossfall has to be eliminated by gradually reversing the gradient. In both cases, super-elevation is applied by rotation about the inner edge.

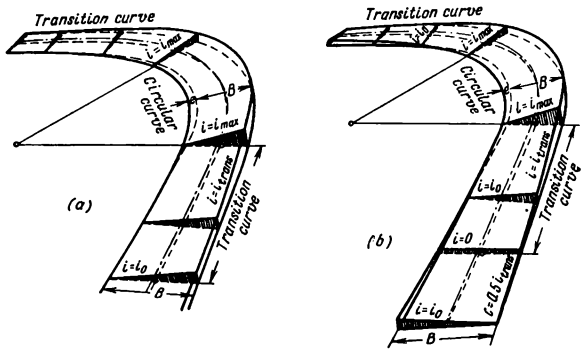


Fig. 43. Diagram showing application of super-elevation to straight fall carriageways:

a—carriageway gradient coinciding with the inclination of the super-elevation;
b—carriageway gradient falls in the direction opposite to the super-elevation, showing elimination of adverse crossfall

The super-elevation on roads with separate carriageways may be applied as a common slope for both carriageways, as well as in two separate parts for each carriageway (Fig. 44). The first method provides an easier way to drain the water from the carriageways, but requires an extensive volume of earthworks whilst the high embankment produced is not pleasing. Because of this, the method most frequently used is to provide two separate carriageway super-elevations. In this case it is difficult to drain the water from the median and it is necessary to provide underground culverts of the same type as in inhabited localities.

On horizontal curves the transverse gradient of the shoulders is made equal to the super-elevation of the main road carriageway. The gradient of the shoulders and verges is altered 10 m before the start of the transition.

The length of the transition to super-elevation should not be too short, since in this case when the vehicle travels at a high speed over a changing transverse road profile the resultant side-sway is uncomfortable for the passengers. The minimum length of the vertical

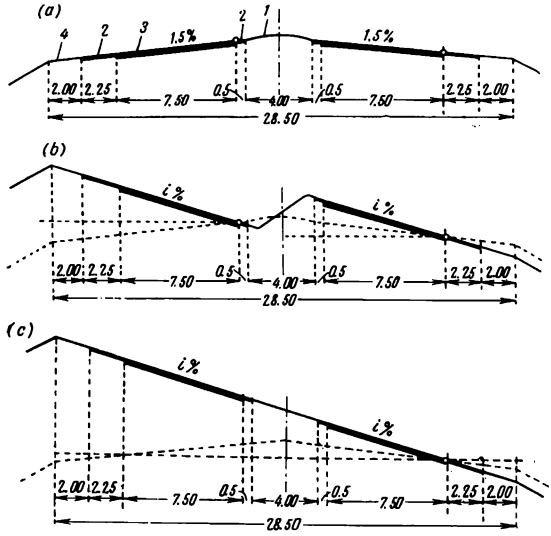


Fig. 44. Application of super-elevation on motorways with separate carriageways:

a—cross-section on a straight; *b*—independent super-elevation for each carriageway; *c*—common super-elevation for both carriageways; *1*—median; *2*—hard strip; *3*—carriageway; *4*—stabilized shoulder

transition is determined by the additional gradient appearing at the outer edge of the carriageway in consequence of its elevation when building the super-elevation.

If the gradient along the road centre line is i, then at the edge the total gradient will be

$$i_{edge} = i + i_{add} = i + \frac{Bi_{tro}}{L} \tag{75}$$

where B = width of the carriageway
L = length of the transition to super-elevation
i_{tro} = transverse gradient of the surfacing.

If a curve is situated on a stretch of road with a steep gradient the magnitude of the longitudinal gradient at the outer edge may exceed the maximum permissible value for the given road. It is imperative that the total gradient at the edge of the carriageway should not be greater than the one permitted for this road in exceptional cases. The additional longitudinal gradient at the transition to super-elevation for roads of classes I and II should have a maximum

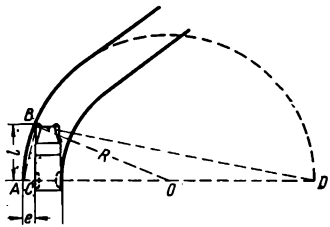


Fig. 45. Diagram showing additional width of traffic lane required on a sharp curve

value of 0.5%, and for other roads a 1% maximum in flat and undulating country, and a 2% maximum in mountainous areas.

Extra width of carriageway on curves. When rounding a bend each wheel of an automobile is instantaneously moving along a separate trajectory, as a result of which the width of the track occupied by a vehicle on a road is increased (Fig. 45). To make the conditions of motion along a curve similar to those along a straight stretch, the carriageway along the curve should be widened. Assuming that the path of motion of the vehicle within the curve is the arc of a circle, it is possible to obtain an approximate expression for the required extra width of the curved traffic lane.

From the similarity of the triangles ABC and BCD we find that

$$\frac{AC}{BC}=\frac{BC}{CD} \quad \text{or} \quad AC\,(2R-AC)=l^2$$

Neglecting the quantity AC within the parentheses, which is small compared to $2R$, we find that the required extra width e for a single traffic lane is

$$e=AC=\frac{l^2}{2R} \tag{76}$$

The deduced formula determining the extra width of the curved path is based on purely geometrical considerations and, therefore,

is valid only for low traffic speeds. At high traffic speeds a comparatively small deviation of the wheels can carry the vehicle beyond the limits of the traffic lane, and for this reason it is recommended that the widening provided should be greater than the calculated one. According to experimental data, the influence of the traffic speed V km/hr on the required degree of widening is given in the following empirical formula:

$$e = \frac{l^2}{2R} + \frac{0.05V}{\sqrt{R}} \tag{77}$$

It is still more difficult to determine the extra width of the road in the case of combination vehicles, in which every trailer moves along its own path. The width of road taken up by the vehicle increases with the number of trailers.

Since the speed of combination vehicles is slow compared to single motor cars, the calculation based on purely geometrical reasoning is more substantiated. On curves of radius less than 700 m the widening required for a carriageway with two traffic lanes should be as follows:

Curve radii, m	700-550	500-450	400-250	200-150	150-90	80-70	60	50	40	30
Extra width, m	0.4	0.5	0.6	0.75	1.0	1.25	1.4	1.6	1.8	2

The extra width of the carriageway is applied on the inside of the curve by replacing part of the shoulder. The width of the formation should be increased only in such cases when the remaining part of the shoulder has a width less than 1.5 m on roads of classes I and II and 1 m on roads of the remaining classes.

Within the limits of the circular part of the curve the widening is constant and reduces gradually to zero over the length of the transition curve.

On mountainous roads, where the curve radii are sometimes as low as 20-30 m, a vehicle with a large wheelbase cannot keep within the inner half of the carriageway. In this case the extra width of a curve must be applied on the outer side. This method is especially expedient for curves situated with their convexity towards the downward slope.

Transition curves. When a vehicle enters a curve from a straight, the conditions of motion suddenly alter since the vehicle now becomes subject to the action of a radial acceleration. Theoretically this effect takes place instantaneously, but in practice it develops over a short length of road along which the driver gradually turns his steering wheel.

To prevent the radial acceleration from building up too rapidly and thus affecting the occupants and perhaps causing a skid, a tran-

sition (easement) curve is introduced between a straight and a curve of short radius. Along this transition curve, the curvature is increased progressively from zero on the straight to $1/R$ at the place where it joins the circular curve.

The paths which individual drivers select when negotiating a bend may differ appreciably, some vehicles departing from the mean line by perhaps 30 to 50 cm. For this reason the setting out of transition curves is based on the diagrammatic representation of the motion of a vehicle along the curve assuming that the velocity at the approach thereto does not vary, and that the driver steers the wheels of the vehicle with constant angular velocity. The driver is assumed to begin turning the steering wheel upon his entering the transition curve.

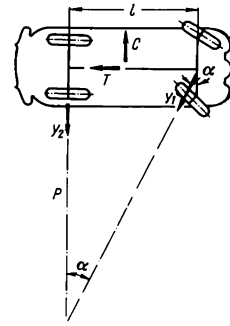


Fig. 46. Diagram for determining vehicle steering radius

The movement of the vehicle within the limits of the curve can be expressed as the result of the combination of two independent motions: a translational with a speed $v = ds/dt$, where s is the length of the distance travelled along the transition curve, and rotary with an angular speed $\omega = d\alpha/dt$.

The angle through which the vehicle wheels are turned (Fig. 46) is

$$\alpha = \text{arc tan}\,\frac{l}{\rho} \tag{78}$$

but since the vehicle wheelbase l is substantially less than its turning radius ρ and, therefore, the angle α is small, one can assume that

$$\alpha \approx \frac{l}{\rho}$$

Then

$$d\alpha = -\frac{l}{\rho^2}\,d\rho \tag{79}$$

Whence

$$\omega = -\frac{l}{\rho^2} \times \frac{d\rho}{dt} \tag{80}$$

Substituting

$$dl = \frac{1}{v}\,ds$$

we obtain

$$\omega = -\frac{lv}{\rho^2} \times \frac{d\rho}{ds}$$

or

$$ds = -\frac{lv}{\omega} \times \frac{d\rho}{\rho^2} \tag{81}$$

For solving the above differential equation of the vehicle transition trajectory it is assumed that the rate of application of angular velocity to the front steering wheel is constant along the transition curve, i.e.,

$$\omega = \text{const} = \omega_0$$

Integration of the differential equation gives the following expression:

$$s = \frac{lv}{\omega_0 \rho} + C \tag{82}$$

The constant of integration C can be determined from the condition that at the point of origin of the transition curve $s = 0$ and $\rho = \infty$.

Then

$$C = 0 \quad \text{and} \quad s = \frac{lv}{\omega_0 \rho} \tag{83}$$

The obtained equation of the transition curve is an equation of a *radioidal spiral* (*clothoid*).

If the constant factor lv/ω_0 is expressed as a coefficient K, the equation of the transition curve can be written as follows:

$$s = \frac{K}{\rho} \tag{84}$$

i.e., the radius of curvature ρ at any point is inversely proportional to the length of the curve s.

At the end of the transition curve $s = L$ (the length of the transition curve), and $\rho = R$ (the radius of the circular curve). Hence $K = LR$, and therefore

$$\rho = \frac{LR}{s} \tag{85}$$

The *length of transition curves* is chosen to permit the gradual building up of centrifugal force and obviating discomfort to vehicle occupants.

The centrifugal acceleration transmitted to the vehicle during its motion along a curve is $I = v^2/\rho$. As the radius of curvature shortens the value of the centrifugal acceleration I increases, and for this reason the length of the transition curve is set out with the purpose

of creating conditions at which the rate of centrifugal acceleration build-up will not exceed the value easily tolerated by the occupants.

Investigation shows that the majority of drivers do not exceed a value of $I = 0.8$ m/sec^2. This value is close to the value of I accepted for calculations.

The time required for the centrifugal acceleration to build up uniformly from zero to v^2/R, is $t = v^2/RI$.

Hence the required length of the transition curve becomes

$$L = vt = \frac{v^3}{RI} \tag{86}$$

or, expressing the speed V in km/hr

$$L = \frac{V^3}{3.6^3 RI} = \frac{V^3}{47RI} \tag{87}$$

Taking the design value of $I = 0.5$ m/sec^2

$$L = \frac{V^3}{23.5R} \text{ metres} \tag{88}$$

In the U.S.S.R. transition curves are designed on roads of all classes for circular curves with radii less than 1,500 metres. Generally the following lengths of transition curves are used:

Radii of circular curves, m	30	50	60	80	100	150	200	250	300	400	500	600-1,000	1,000-2,000
Length of transition curves, m	30	35	40	45	50	60	70	80	90	100	110	120	100

Usually the transition curve is combined with the transition to super-elevation. If according to the calculation the transition to super-elevation would be longer than the length of the horizontal transition curve, the latter should be increased.

When laying out highways designed for high-speed traffic the transition curves become in themselves major horizontal and vertical design elements of equal importance with straights and curves, instead of being an auxiliary component for planning curves of short radius. Long transition curves ensure the required smoothness of the route, not only from the point of view of vehicle stability and convenience for traffic, but also for the driver's visual perception. This aspect is developed below in Sec. 62, which deals with special aspects of highway landscape engineering.

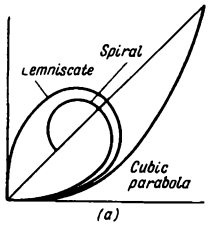
Types of transition curves. The most rational contour of a transition curve is the radioidal spiral (radioid) or the clothoid, whose radius of curvature at any point is inversely proportional to the length of the curve from its origin.

In the practice of road building, a number of other curves additional to the clothoid are in use (Fig. 47*a*):

(a) a *cubic parabola*, which is a curve having a radius of curvature proportional to its abscissa;

(b) *Bernoulli's lemniscate*, which is a curve whose radius of curvature is proportional to the length of its chord;

(c) a *compound curve*, which consists of separate segments of circular curves (Fig. 47*b*).



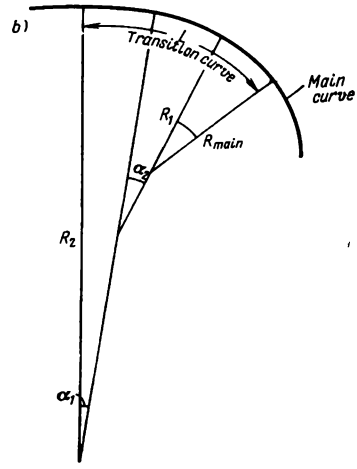


Fig. 47. Transition curves:
a—lemniscate, spiral, cubic parabola; *b*—compound curve

When setting out a three-centered compound curve, the radius of each offset is made twice the length of that of the following segment of the curve.

The difference in ordinates of the transition curves set out according to various formulas, is often well within the limits of accuracy of possible motor vehicle deviations from the average path. For this reason the equation of the transition curve is usually chosen with the aim to facilitate its setting out. The method most commonly used is the setting out of transition curves along a clothoid.

The equation of the latter in a system of rectangular coordinates has the form

$$x = s - \frac{s^5}{40c^2} + \frac{s^9}{3{,}456c^4} - \dots \tag{89}$$

$$y = \frac{s^3}{6c} - \frac{s^7}{336c^3} + \frac{s^{11}}{42{,}240c^5} - \dots \tag{90}$$

where $c = RL$, and s is the length of curve segment corresponding to the coordinates x and y.

The series of x and y converge rapidly, and for compiling tables usually only the first two members of equations (89) and (90) are used.

The introduction of the transition curves calls forth shifting of the main circular curve towards the centre of the bend (Fig. 48). This has to be taken into account when setting out the angles of turn and the radii of curves, since the length of the bisector is increased by the extent of the shift of the curve inside the angle

$$p = y_0 - R(1 - \cos\varphi) \quad (91)$$

where y_0 = ordinate of the transition curve at the point of its junction with the circular arc

$\varphi = L/2R$, rad.

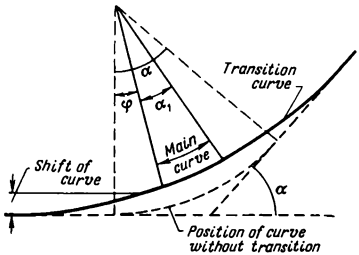


Fig. 48. Diagram for setting out transition curves

Part of the circular curve has been replaced here by the transition curve. According to Fig. 48, the setting out of the transition curve is possible on condition that $2\varphi \leqslant \alpha$. If this condition is not satisfied, the length of the transition curve should be reduced or the radius R increased.

24. Provision of Visibility on Curves

From safety considerations the driver should be able to see the road in front of him for a distance sufficient to notice an obstruction, realize its danger, and have time to avoid it or pull up.

When designing a road a safe vision distance has to be secured, i.e., a distance in front of the vehicle at which the road should be visible to the driver. This is known as the *minimum sight distance.*

Methods of determining the minimum sight distances have been proposed, relating vehicle/driver conditions to vehicle spacings, as well as to the location of obstructions on the road. All these schemes can be divided into two groups differing according to their principle:

(a) The application of brakes to bring the vehicle to a standstill in front of the obstruction or an on-coming vehicle.

(b) The execution of vehicle manoeuvre, based on the expediency of avoiding the obstruction by the vehicle, of overtaking a car moving

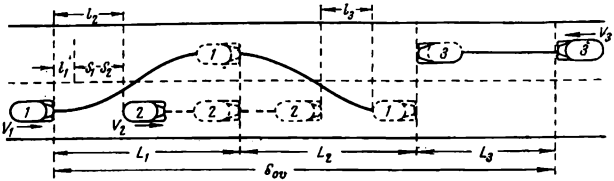


Fig. 49. Sight distance required for overtaking

in the same direction, or passing a vehicle proceeding in the opposite direction after having drawn out into the opposing traffic lane. These schemes are used mainly for estimating the minimum sight distances on vertical curves.

In the first case—the one most commonly used and on which recommendations for technical standards are usually based—the same pattern of braking is envisaged as for calculating road lane capacity (Sec. 18)

$$S = \frac{V}{3.6} + \frac{KV^2}{254(\varphi \pm i + f)} + l_0 \text{ metres} \tag{92}$$

where V is the speed in km/hr.

Depending on the initial assumptions, the stopping of the vehicle in front of an obstruction, or the movement of two vehicles in opposite directions on the same lane, can be analysed.

In the case of overtaking, the most desirable method for computing minimum sight distances is shown in Fig. 49. This presupposes the following sequence of actions.

Overtaking commences when the overtaking vehicle *1* approaches the front vehicle until the interval equals the difference of their stopping distances ($S_1 - S_2$). Before drawing out into the opposite traffic lane a certain time elapses from the moment when the driver makes the decision to overtake, which—similarly to the process of braking examined previously—may be assumed equal to 1 sec. During this time the vehicle would cover a distance $l_1 = V_1$ m. Therefore, overtaking starts at a distance from the slow-moving vehicle of

$$l_2 = V_1 + (S_1 - S_2) = V_1 + \frac{K(V_1^2 - V_2^2)}{2g\varphi} \tag{93}$$

where V_2 = speed of the front vehicle, m/sec
V_1 = speed of the overtaking vehicle, m/sec
φ = coefficient of adhesion
K = coefficient of operational efficiency of the brakes.

The speed difference of the two vehicles being $(V_1 - V_2)$, to close this gap the overtaking vehicle will travel a distance

$$L_1 = \frac{l_2 V_1}{(V_1 - V_2)} = \frac{V_1^2}{(V_1 - V_2)} + \frac{KV_1(V_1 + V_2)}{2g\varphi} \tag{94}$$

Having drawn level with and passed the slow-moving vehicle, the overtaking vehicle now has the opportunity to return to the nearside lane. In accordance with safety requirements the place of return should be situated at a distance from the position of the overtaken vehicle equal to the latter's stopping distance S_2, increased by a certain safety margin $l_0 = 5$ to 10 m

$$l_3 = \frac{KV_2^2}{2g\varphi} + l_0$$

Hence the distance travelled by the vehicle *1* when drawing back to the nearside traffic lane is

$$L_2 = \frac{l_3 V_1}{V_1 - V_2} = \left(\frac{KV_2^2}{2g\varphi} + l_0\right)\frac{V_1}{V_1 - V_2} \tag{95}$$

The extreme case of the possibility of overtaking with the necessity of drawing out into the opposing traffic lane corresponds to the return of the overtaking vehicle to its nearside lane at the moment when it draws level with a vehicle travelling in the opposing direction. During the period of overtaking this vehicle will travel a distance

$$L_3 = \frac{(L_1 + L_2)V_3}{V_1} \tag{96}$$

Hence the safe sight distance based on conditions of overtaking is

$$S = L_1 + L_2 + L_3 = \left[\frac{V_1^2}{V_1 - V_2} + \frac{KV_1(V_1 + V_2)}{2g\varphi} + \right.$$
$$\left. + \left(\frac{KV_2^2}{2g\varphi} + l_0\right)\frac{V_1}{V_1 - V_2}\right]\left(1 + \frac{V_3}{V_1}\right) \tag{97}$$

There is a series of similar formulas, assuming certain simplifications or complications of the described overtaking process. All these are based on various assumptions concerning the relation between the relative speeds of two vehicles and their operating conditions. The results of determining minimum sight distances according to these formulas differ appreciably from each other.

In practice, the majority of countries provide for a minimum safe sight distance of an order of 300 to 400 m, derived for the case of braking in front of an obstruction.

The minimum sight distances ensured on U.S.S.R. highways are given below.

Name of visible objects	Minimum sight distances, m, for roads of class				
	I	II	III	IV	V
Surface of road:					
(a) in ordinary conditions	250	175	140	100	75
(b) on difficult sections of broken country	175	140	100	75	50
(c) on difficult sections of mountainous terrain	100	75	60	50	40
Opposing vehicle:					
(a) in ordinary conditions	—	350	280	200	150
(b) on difficult sections of broken country	—	280	200	150	100
(c) on difficult sections of mountainous terrain	—	150	120	100	80

In all cases when this does not lead to a noticeable increase in construction costs it will be good practice to ensure a sight distance of at least 350 m on roads of all classes.

On motorways of the F.R.G. the minimum permitted sight distance is from 300 to 150 m. However, in practice the designers seek to provide a visibility of not less than 750 m, which allows for safe overtaking.

The visibility on horizontal curves should be checked for a vehicle proceeding along the inner nearside traffic lane. It is usual to assume that the driver's eye is situated 1.5 m from the edge of the carriageway and at a height of 1.2 m. This corresponds to the position of a driver of a passenger car. Since the safe visibility distance is considered to be the distance which the vehicle covers before pulling up in front of an obstruction on a road, the test sight distance is measured along the path of the vehicle.

Theoretical investigation into visibility along horizontal curves makes it possible to derive a mathematical expression relating the driver's range of vision to motion along a curve.

In practice it is usual to determine the boundary of the zone of visibility where obstacles have to be set back by the *graphical method.*

On a large-scale plan of the bend (Fig. 50), a series of points are plotted representing the positions of a vehicle travelling along the road. From these points, lines are drawn of length equal to the minimum sight distances (1-1′, etc.). The curve inscribed along the inner parts of these distances is the boundary of visibility.

In plotting the level of setting back it is necessary to take into account that in future the clearings may be covered by grass or

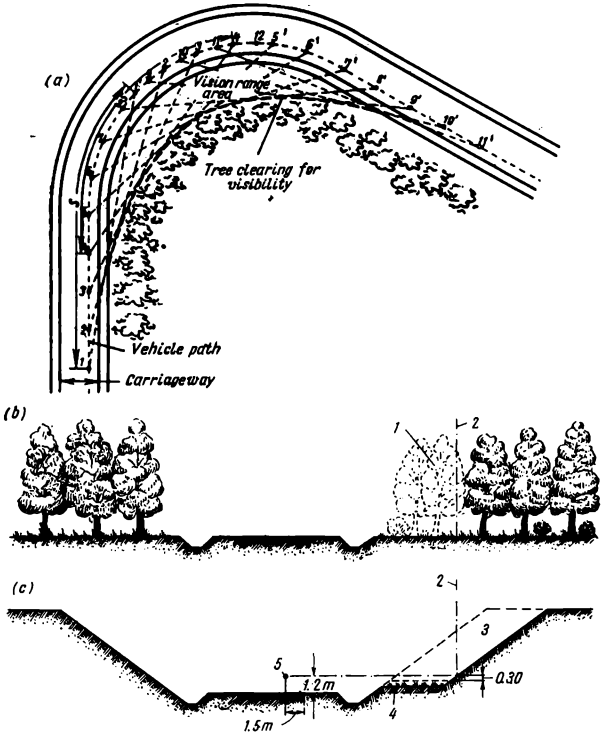


Fig. 50. Provision of sight distance on horizontal curves:
a—graphic location of boundary, set back to provide visibility; *b*—boundary of tree clearing; *c*—limit of set-back inside a cutting; *1*—clearing for visibility; *2*—boundary of visibility zone; *3*—most expedient level of set-back; *4*—minimum level of set-back; *5*—position of driver's eyes

snow. It is therefore more expedient to carry cuttings down to the level of the road surface.

To check the provision of safe sight distances on plan and to simplify plotting of the boundaries of the zones of visibility it

will be sufficient to determine the extent of set-back in the middle of the curve, along the bisector.

Let us consider the general case when the length of the curve K is less than the required safe sight distance S.

According to Fig. 51, the required set-back is

$$\delta = DE + EH \tag{98}$$

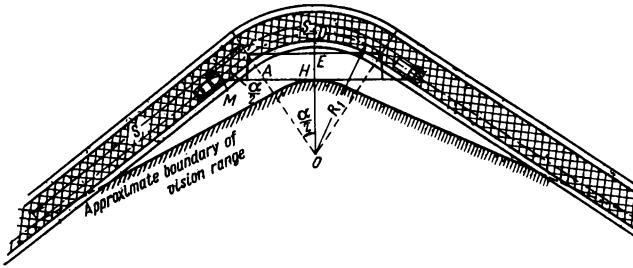


Fig. 51. Determination of set-back to clear vision range

In this expression $DE = R_1 - OE$, where R_1 is the radius of the vehicle path. But $OE = R_1 \cos \frac{\alpha}{2}$, where α is the angle subtended at the centre of the curve. Hence

$$DE = R_1 \left(1 - \cos \frac{\alpha}{2}\right) \tag{99}$$

$$EH = AF = FM \sin \frac{\alpha}{2}$$

Since

$$FM = \frac{1}{2}(S - K) = \frac{1}{2}\left(S - \frac{\pi R \alpha}{180}\right)$$

then

$$EH = AF = \frac{1}{2}\left(S - \frac{\pi R \alpha}{180}\right) \sin \frac{\alpha}{2} \tag{100}$$

and the total extent of set-back is

$$\delta = R_1 \left(1 - \cos \frac{\alpha}{2}\right) + \frac{1}{2}\left(S - \frac{\pi R \alpha}{180}\right) \sin \frac{\alpha}{2} \tag{101}$$

In the particular case when $K > S$, expression (101) is simplified and becomes

$$\delta = R_1 \left(1 - \cos \frac{\alpha_1}{2}\right) \tag{102}$$

where α_1 is the angle with a length of arc equal to the sight distance and constituting

$$\alpha_1 = \frac{S\,180}{\pi R_1} \text{ degrees} \tag{103}$$

In both cases the set-back within the limits of the curve can be assumed constant its boundary can be traced along a concentric circumference. The set-back should be started on a straight or on a transition curve at a distance from the start and the end of the curve equal to the sight distance S.

If the visibility is limited by girders of bottom-road bridges or overpass supports, in the majority of cases it is advisable to amend the alignment of the road, by eliminating the curve or substantially increasing its radius.

25. Standard Conditions for Road Design

Speed-time-distance calculations make it possible to establish the requirements for horizontal and vertical motor highway elements corresponding to each given type of vehicle. The roads, however, carry a variety of vehicles with different loading and various degree of usage, and which are driven by drivers of differing temperaments and driving experience. Hence, for practical guidance in road designing geometric layout standards are defined, i.e., engineering conditions based on speed-time-distance calculations for average conditions of traffic. The standards used for technical specifications for roads of various classes take into account the dynamic qualities of most popular vehicles and the cost of road construction in various topographic conditions. For economic reasons a certain waste of dynamic qualities of vehicles is tolerated on roads of lower classes having low traffic flows, and in difficult topographic conditions.

Since the engineering specifications are to be valid for a number of years, they must take into consideration the probable effects of vehicle improvement, and so determine the technical policy in the field of road construction.

The need for ensuring the most rational use of the capital invested in road construction makes it necessary that the engineering conditions permit the steady improvement of roads as traffic volumes rise, coupled with road building in stages.

The establishment of engineering specifications for motor road design in the U.S.S.R. is aimed at rational and safe operation of motor transport, comfort for road users, and at facilitating the work of drivers.

Along with the theoretical calculations used for establishing technical specifications for road engineering, wide use is made of

the experience gained from earlier road constructions, since the initial data, e.g., the design speed, the coefficient of adhesion, drivers' eye height and road hazards, are determined according to the analysis of road operating conditions. Great importance is attached to the investigation of road accidents due to road conditions, particularly through the unsuccessful combination of horizontal and verfical road elements.

Specifications for design of road works have been repeatedly revised as numbers and varieties of vehicles have increased, with standards becoming more and more exacting.

However, one should not consider that the work of drawing up standards is over. Natural conditions in various countries differ widely. Therefore, the road operating conditions which are typical of the north (excessive moistening of subgrade, marshy ground or likelihood of winter ice covering) are quite alien to southern regions. On the other hand, in south-eastern regions where there is an arid climate, a highly saline soil or loose sands, etc., road engineering has to satisfy specific requirements not concerning highways in the northern regions.

Therefore, for further refinement of road design standards, it is necessary to establish quite independent specifications for road layouts in typical areas, a common element for which should be a constant design speed for roads of various classes which can be maintained over sufficiently long periods of time during the course of a year.

This requires a thorough study of natural and climatic conditions in order to assess the rated value of the coefficient of adhesion between tyres and road surfacing for the chosen road elements.

The values of the coefficient φ used should correspond to the most typical unfavourable weather conditions for each natural region. In some regions, when determining the width of the carriageway—especially for roads of a lesser technical class—it is necessary to consider the movement of special types of agricultural and carrier machines. For regions with a uniform and clearly seasonal distribution of traffic throughout the year, various methods for determining rated traffic intensity will be applied. Details of water-temperature conditions of subgrades and pavements make it possible to relate specifications and constructional standards to specific natural regions.

In view of the high cost of road building in undulating country and highlands the road designers should consider, along with the achievements in the field of traffic and highway theory, design and operation, the reverse problem, namely, the best possible adaptation by manufacturers of vehicles to the typical conditions of natural areas.

PART III

Design of the Roadbed and Pavement

CHAPTER 5

NATURAL FACTORS AFFECTING ROAD PERFORMANCE

26. General

A highway is subject to the action of numerous natural geophysical factors, the most important of which are the climate and hydrological conditions, in addition to the soil texture and geological structure of the country, its topography and vegetation.

It is often difficult to segregate the influence of separate natural factors on the road. Apart from their direct action on road construction or operational conditions, each of them is interlinked with the others, weakening or intensifying their action. Thus, the topographic features which determine to a large degree the longitudinal gradients, the length of the road, and the volume of earthworks required for road building, also influence the volume of water seepage towards the road, the hydrogeological conditions of the country, the soil covering and the nature of the vegetation. Therefore, the natural conditions for a region where the route is to be laid should be assessed as a composite unit, and applied to various landscape zones characterized by certain combinations of natural factors, and by the likelihood that these may be altered radically as a result of human activity. Thus, the clearing of forests leads to drying out of soil, and perhaps to erosion on an extensive scale. Conversely, artificial irrigation often elevates the water table and makes the climate milder.

The *regional topography* influences the magnitude of longitudinal gradients when determining the alignment of a route and the extent of departures from the most direct route to overcome slopes and to bypass boggy or flooded districts. Thus the topographic features

determine the volume of water flowing towards small bridges and culverts. The water and temperature conditions of the roadbed in highlands and highly broken land depend on the degree of severity of the slope on which the road is laid. During the building of the road one has to take into account the local topographic features when deciding on the method of executing the earthworks and when designing the roads for the passage of heavily laden trucks.

The influence of topographic conditions on the design solutions adopted is reflected through the operational cost of transportation and the effect of the actual geometric design on vehicle performance. Steep gradients sometimes make it necessary to limit the loads on transport vehicles, they increase fuel consumption and can be dangerous for traffic when the surface is muddy or ice-covered.

According to the road design and building problems the topographic features can be divided into three types:

(1) *Flat country* and *gently undulating ground*, consisting of gently sloping areas divided by river valleys, occasional ravines and watersheds, with individual and infrequent hills. In these regions the total length of the road stretches with the maximum gradients does not exceed 10 per cent of the route length.

(2) *Broken country*, consisting mainly of hilly ground with narrow watersheds and an appreciable number of ravines, and of foothills with moderately rugged features. The extent of the road stretches with the maximum gradients may vary from 10 to 20 per cent of the total route length.

(3) *Mountainous regions* typified by slopes of mountains and hills having highly rugged topography, by mountain river valleys forming tortuous deep gorges, and passes requiring the construction of reverse loops. The total extent of road stretches having the maximum longitudinal gradients will be over 20 per cent of the route length.

The *geological conditions* define the degree of ground stability in the vicinity of the selected route (Fig. 52).

In the case of unstable surface deposits (landslides, talus and karst cavities) one has to provide for special measures in the project to ensure the stability of the roadbed and structures, or re-route the road through another and more stable district.

A geological survey will establish the existence of local road-building materials, i.e., stone, sand and gravel, which should be preferred for the building of the road pavement.

When choosing the direction of the route one has to consider also the mantle of soil—the "drift"—on the Geological Survey Map. At the reconnaisance stage attempt should be made to bypass highly swamped and saline regions, and regions of loose sands blown about by wind, provided that this does not result in a substantial lengthening of the route. The results of the particle-size analysis will

determine the required elevation of the formation and the depth of the drain ditches. To select the most desirable form of pavement construction it is necessary to take into account the permissible bearing power of soils (deformation modulus). When planning and performing earthworks it is necessary to take into account the difficulty of working the soil, as this will influence the productivity

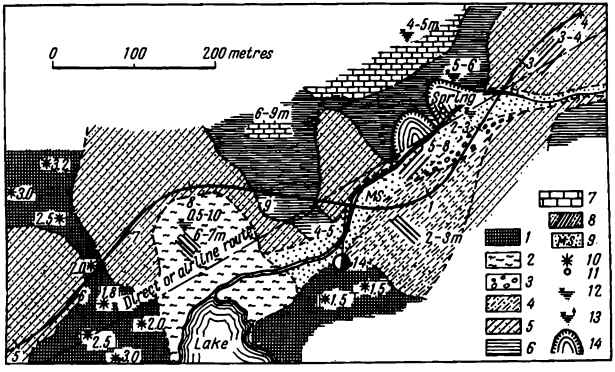


Fig. 52. Effect of geological structure on route location:

1—well decomposed peat of average density; *2*—silty lake deposits, unstable under load; *3*—gravel; *4*—mantle of sandy loam (drift); *5*—moraltnic loam; *6*—heavy stratified clay; *7*—limestone; *8*—method of indicating rock lying below the drift, and their depth of occurrence; *9*—medium sand; *10*—places of swamp probing and depth of bed; *11*—ground-water discharge, as springs; *12*—depth of occurrence of ground water; *13*—depth of occurrence of artesian water; *14*—folds

of the machinery. The soil and the hydrogeological conditions will determine to a large extent the possibility of road damage due to heaves, and the erosion of roadside and drainage ditches by water.

The worthiness of earth roads depends to a great extent on the granulometric composition of the soil composing them: it is exceedingly difficult to drive along sandy ground in dry weather, while clay soil becomes saturated during the wet seasons and dries out slowly when dry weather returns.

Climatic conditions exercise a very strong influence on the operational performance of roads. Among these are the amplitude and the frequency of temperature variations, the maximum and minimum

temperatures, precipitation and evaporation, wind direction and strength, the depth of snow covering and the depth of frost penetration.

These factors must all be taken into account when designing the roadbed. Climatic conditions often limit the length of the construction season, or require the use of special techniques which complicate the operations. Snowfalls and blizzards may interrupt road traffic. An ice covering, by lowering the coefficient of adhesion between the pneumatic tyre and the road, will enhance the possibility of vehicle accidents. In hot climates, where there are periods of prolonged rainfall and drought seasons, the extreme variations in moisture content of the subgrade may cause severe shrinkage and the destruction of the road pavement.

To predict the influence of climatic conditions in the vicinity of the road at various seasons, it is usual to produce a graph of climatic conditions (Fig. 53), on which the yearly temperature variation, the quantity of rainfall, the height of snow covering, etc., are plotted. Figure 53 shows a graph for the European part of the U.S.S.R. Knowing the temperatures at which various road works can be carried out, one indicates on the diagram of climatic conditions, by means of horizontal lines, the periods suitable for these works. When determining these periods one should bear in mind that the influence of climatic conditions decreases with the continued growth of mechanization and adoption of industrial methods, as well as the use of new road-building techniques and the improvement of road technology. To plan the number of working shifts and to determine the periods when artificial illumination of the works will be necessary it is advisable to mark on the diagram the length of the daylight period.

The hydrological and hydrogeological conditions are defined by the quantity of rainfall, the conditions of runoff and evaporation of water, the depth of occurrence of soil water, its movement and retention, and the hydrological conditions of rivers and streams. All these conditions must be taken into account when determining the nature and amount of road drainage for deciding on the construction of the roadbed.

When appraising the influence of the natural factors on the motor road operational conditions one should also consider the reverse action, i.e., the alteration of natural conditions in the vicinity of the road as a result of the road construction. Thus, the felling of trees along the final route and the clearing of roadside vegetation will contribute to partial dewatering of the site. On the other hand, the crossing of a swamp by an embankment, which compresses the peat, may prevent the seepage of soil water and so encourage bog formation.

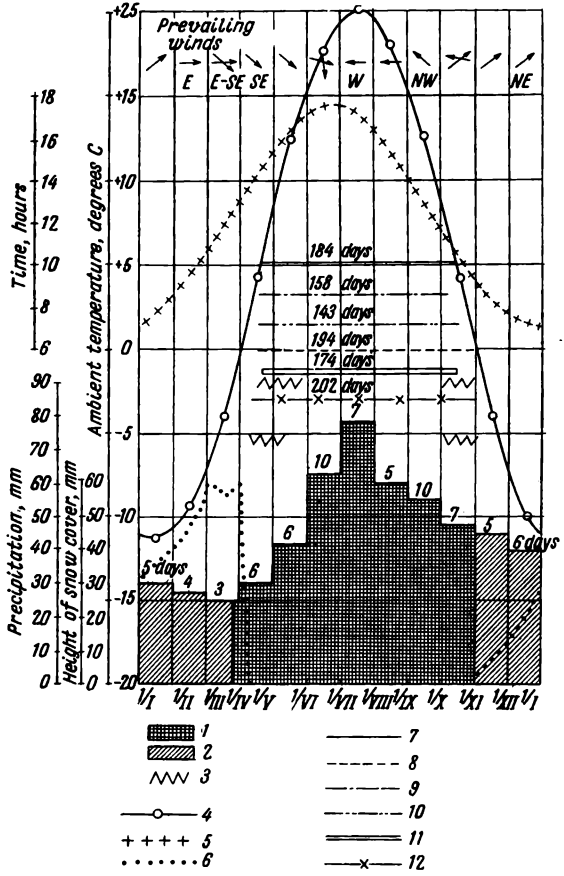


Fig. 53. Graph of physical and climatic conditions in the construction area:

1—rainfall; *2*—snowfall; *3*—earth roads impassable; *4*—temperature; *5*—daylight duration; *6*—thickness of snow cover; *7*—earthworks and roadbed construction; *8*—construction of minor bridges and culverts; *9*—construction of asphalt concrete pavements; *10*—construction of light-weight stabilized pavements; *11*—construction of cement concrete pavements; *12*—working of quarries next to route

27. Factors Causing Saturation of the Roadbed

In dry summers earth roads have a solid, even surface which is suitable for traversing by motor vehicles at high speeds. However, after rain the soil will have absorbed the water and become soft; in this state it is easily ploughed through by the wheels, forming deep ruts. When this happens, wheel drag increases, the coefficient of adhesion between the tyre and the ground surface decreases, and often motor traffic becomes hopelessly bogged down until the road dries out.

Roads which have metalled surfacing made from strong stone materials—*a pavement*—are also often destroyed by traffic if the subgrade becomes saturated with water. Steep slopes of embankments and cuttings may slide after becoming saturated with water. The saturation of the roadbed with water is very dangerous because it causes an appreciable reduction in the stability of the basic elements of the roadbed. The prevention of subgrade saturation is complicated by the fact that the water penetrates into it by two different ways, i.e., by seepage down from the road surface and by elevation from the water table.

Rain water partly runs off an earth surface and partly percolates downwards, accumulating in the pores of the soil above the impermeable layers near the surface. The upper surface of the resultant retained soil water which saturates the pores of the lower part of the permeable layer is called the *ground-water table.*

The ground-water table tends, in general, to follow the smoothed contours of the land. Usually it is elevated slightly under the hills and is depressed under the valleys.

In places where the ground-water table intersects the surface, springs and marshes may occur. In such places the water table is depressed forming a curved surface, called a *depression curve.* The less permeable the ground, the steeper is the depression curve.

The interstices between the earth particles are very small and form thin irregular, tortuous and sensibly continuous channels—the capillaries. The water moves along these capillaries both from the ground-water table and from water remaining after rainfall on the surface of the ground. In this way, above the water table and in the upper soil layers there occur two zones of capillary water after rainfall: one rising from the water table and another, the perched water table, which is not related to ground water but is formed next to the surface after rainfall.

The elevation attained by water rising owing to capillary forces along the capillaries from the ground-water table is dependent on the size of the soil particles and on the degree of the soil compaction. In sands the height of the capillary rise is small (less than 30-50 cm),

in loosely compacted powdery soil, however, it may attain several metres.

The level of the capillary rise corresponds to the level of the ground-water table. The layer of soil above the capillary rise contains water in the form of very thin pellicles, which are measured by fractions of a micron (bound water), and also as vapour contained in the pores between the particles.

The separate kinds of ground water under the roadbed do not remain in static equilibrium throughout the year. Under the influence of water inflow, and also due to alteration of temperature and atmospheric pressure, there will occur variations in the level of the ground-water table, and in the height of the capillary rise, as well as the transfer of water vapour and film moisture from places with a higher temperature to places with a lower one.

Sources of roadbed saturation are rainfall, rain water flow from the higher side in sloping ground, the capillary rise from the ground-water table, the condensation of water vapour in the air and the transfer of film moisture on the surface of soil particles.

Depending on the climatic and local conditions, and on the season of the year, one or the other causes of roadbed saturation may prevail.

28. Water Conditions under the Roadbed

The amount of water W in the roadbed does not remain constant throughout the year and varies in a finite time period in accordance with the pattern of water flow

$$W = \underbrace{(A + B + C)}_{\text{(water inflow)}} - \underbrace{(D + E + F)}_{\text{(water outflow from the roadbed)}} \tag{104}$$

where A = rainfall over the roadbed

B = water inflow from the land adjacent to the road

C = water inflow from the ground-water table along the capillaries, and also by the transfer of film and vapourized moisture

D = water discharge from the roadbed

E = water evaporation from the surface of the soil

F = water seepage from the roadbed into the soil layers.

At different times of the year the correlation between the elements of the above equation alters.

Recent investigations have made it possible to elucidate the movement of water and clarify the picture of moisture cyclic variation in various layers of the roadbed throughout the year.

Apart from rainfall, the water conditions are influenced extensively by the temperature variation throughout the year. This creates,

within the roadbed, thermal gradients, under the influence of which the ground water moves from the warmer places towards the cooler ones.

The character of the variation in water conditions depends substantially on the local climatic conditions, since the influence of these factors constituting the pattern of water flow is different in various climatic zones.

Figure 54 shows the variation in the part played by sources of roadbed saturation as one travels from the north-west to the south-east of the European part of the U.S.S.R.

Since, as one moves south, the depth of the ground-water table grows, the rainfall decreases and evaporation is more intensive, the roadbed water conditions become more favourable. This is enhanced by the fact that simultaneously with the reduction in the intensity of saturation the effect of moisture displacement owing to winter temperature influence also decreases. Therefore, in the southern arid zones the part played by the ground water in the alterations of the water balance decreases, and that played by water vapour transfer grows in importance. The most important sources of roadbed saturation in the region of the plains are rainfall and the condensation of water vapour in the pores of the soil. Correspondingly, in the northern zones the effect on the degree of roadbed saturation of the capillary rise in the ground water situated next to the actual surface becomes more significant, and the effect of the evaporation decreases.

There are several ways of achieving moisture transfer in the soil under the influence of a temperature gradient over a vertical section.

The flow of moisture along the film enveloping the soil particles. All the soil particles are surrounded by water pellicles, bound to their surfaces by molecular forces. The nearer the water molecules are situated to the surface of the soil particle, the greater is the force retaining them. Conventionally, this water is divided into two layers, firmly bound water whose characteristics approach those of solid matter, and the loosely bound water which is capable of flowing from one particle to another under the action of molecular forces. The surface energy of a soil particle which retains around itself the bound water is

$$A = \alpha S \tag{105}$$

where α = surface tension on the soil-water interfaces, dyne/cm
S = surface of the soil particle, cm^2.

Since the surface tension of the water increases with a drop in temperature, a soil particle with a lower temperature can retain a thicker pellicle of bound water than a warmer particle. The water film on a particle which is cooling thickens by drawing in

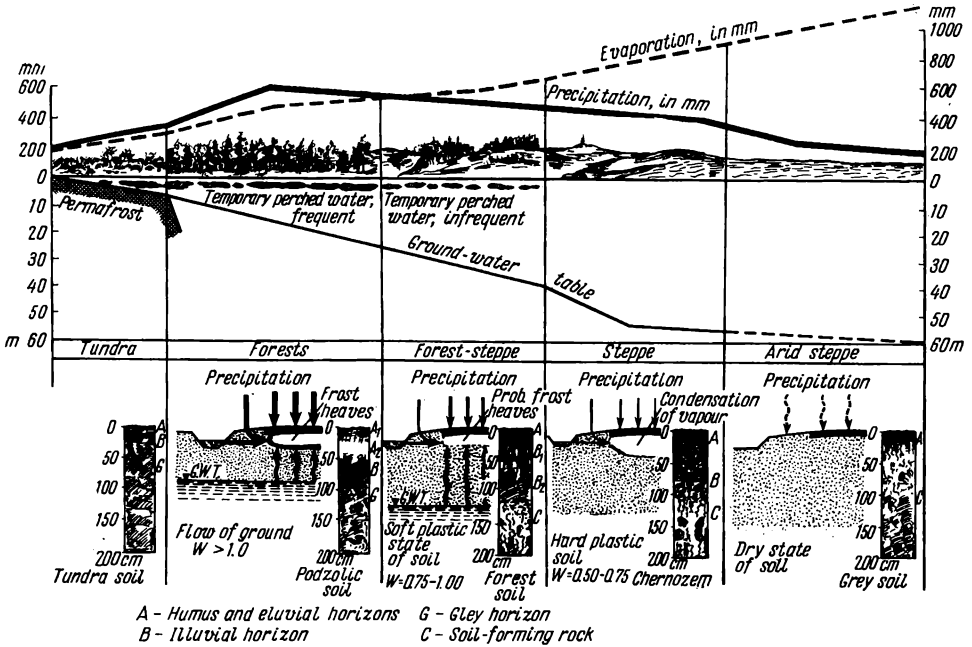


Fig. 54. Relation between climatic factors and roadbed saturation in the European part of the U.S.S.R.

water molecules from the lower warmer layers of soil owing to the system of continually intercommunicating pellicles.

The transfer of moisture by means of water vapour condensation on the surface of cooled soil particles. When a soil is only partially saturated, the circulation of air and of water vapour within the interstices of the soil can take place. During the cooling of the ambient air warm air saturated with water vapour rises up from the ground-water table. The amount of vapour in fully-saturated air decreases with a drop in temperature. Therefore, as the rising air cools off, the water vapour condenses on the surface of the soil particles.

The flow of vaporized moisture decreases as the soil moisture content grows, because the separate capillaries become sealed by menisci rings. This stops circulation when the moisture content reaches a value close to the capillary moisture capacity.

Flow of moisture along capillaries. The cooling of the soil increases the water surface tension and, consequently, the lifting force of the menisci. Because of this the height of capillary rise in the cooling soil will increase somewhat.

The process of moisture flow and its accumulation occurs most intensively in powdery soils where a large proportion of particles are within the 0.5 to 0.002 mm size. In these soils the surface of soil grains is sufficiently developed to retain a thick water film; simultaneously, a flow of vaporized moisture can take place through the soil pores.

In grounds having a large clay content the water flows slowly, since in very thin pores of argillaceous soil the viscosity of the bound water is very high and the soil microstructure offers appreciable resistance to water flow.

29. Demarcation of Road Zones

The conditions of water movement and retention in the roadbed depend on the local climatic conditions in the vicinity of the road itself. The variety of climatic, ground and hydrologic conditions in countries with a vast territory does not allow the road foundations and pavements in their various geophysical zones to be designed according to common rules.

A constructed highway, being exposed to the action of diverse climatic factors, takes part in all the natural processes affecting the upper layers of the earth crust. Though the influence of these natural factors is substantially reduced by such design measures as the elevation of the sub-base, the building of waterproof surfacing, etc., the intensity of their action does not differ from that of the factors acting on the ground surface in this climatic region. The

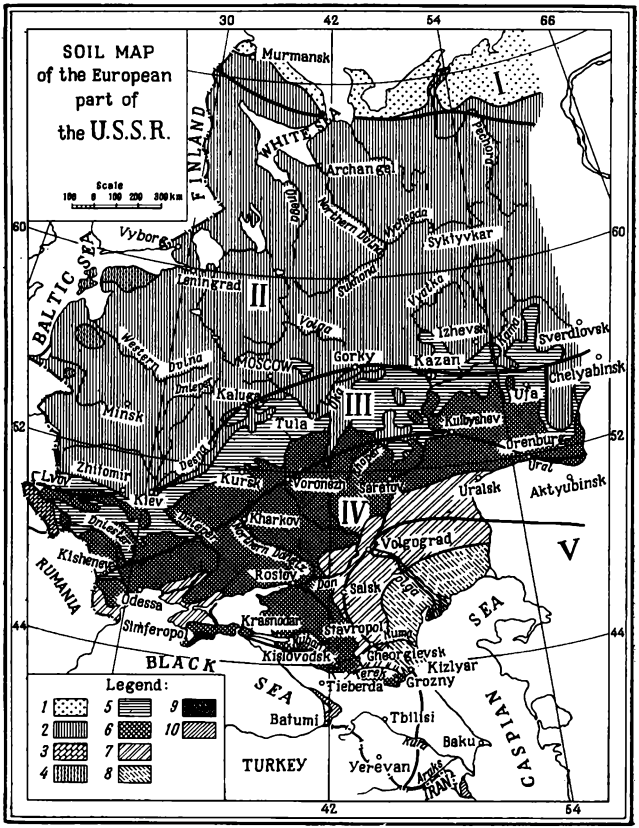


Fig. 55. Comparison of road zones and soil formation of the European part of the U.S.S.R.:

1—tundra soils; *2*—podzol and swamped soils; *3*—mountain and wood podzol soils; *4*—fibrous humus-calcareous soils mixed with podzol ones; *5*—grey forest soils and other soils of forest-steppe; *6*—chernozem soil; *7*—chestnut and alkali soils; *8*—brown soils, alkali soils and occasional sands; *9*—brown soils of southern deciduous forests of the Crimea and Caucasus; *10*—red soil, yellow soil and subtropical podzol soils

principles of road demarcation according to climate worked out for the U.S.S.R. can be taken as a basis for regional demarcations in other countries.

The demarcation of road zones is based on the local hydrologic and temperature conditions, which are characterized by the present mantle of soil and reflect to a certain extent the hydrologic and temperature conditions of ground strata, since the prevalence of certain soil types is indicative of definite climatic zones. The demarcation of a territory into road zones may be initially based on the country's soil map, on which certain corrections can be made with a view to previous road experience. The soil map is used, since the soil types are a product of the action of rain, frost, sun, wind, etc., on the exposed rocks for many centuries.

Comparing the map of road demarcation zones with the chart showing the distribution of soil types (Fig. 55) and with the map of landscape zones for the territory of the U.S.S.R. (Fig. 56, after L. S. Berg), one may see that the zones of road demarcation—defined in accordance with experience of road operation—roughly coincide with the boundaries of soil and landscape zones.

In accordance with road and climatic conditions the territory of the U.S.S.R. is divided into five zones.

Zone I is the zone of persistent frozen ground (permafrost). This zone comprises the zones of tundra, forest-tundra and the north-eastern part of the forest zone, and includes the regions of persistent frozen ground which are notable for the oversaturation of top layers of soil. Deep infiltration of water is obstructed by the frozen ground near the surface. The evaporation is insignificant because of the short summer season.

Within the confines of the European part of the U.S.S.R. the tundra soils prevail throughout Zone I. In view of an obviously apparent relationship between the depth of occurrence of the permafrost layer and the vegetative cover, the topography and the orientation of slopes in relation to cardinal points, there are no typical recommendations for the road foundation and pavement construction in this zone.

Zone II is the zone of excessive ground water. The southern boundary of the zone in the European part of the U.S.S.R. roughly corresponds to the northern boundary of grey forest soils and of the forest landscape zone. In its western part the zone includes the swampy territory of woodlands.

Zone II is notable for an excess of surface and ground water because of the intense rainfall, reduced evaporation and the high ground-water table.

The coefficient of water balance (the ratio of annual absorbed rainfall to the evaporations for the same period) for Zone II varies from 1.5 to 2. Taiga and mixed forests, and podzol soils occur almost exclusively through the zone.

Zone II covers a considerable portion of the territory of the U.S.S.R. comprising regions with sharply differing climatic conditions. No doubt, in the future, it will have to be divided into separate subzones.

Zone III is the one of variable moisture content. The southern boundary of the zone corresponds to the northern boundary of chernozem soil. According to the soil types Zone III is situated within the limits of podzolized soils of the forest-steppe zone and of leached chernozem. The zone is characterized by excessive moisture content in spring and autumn, while the average annual coefficient of water

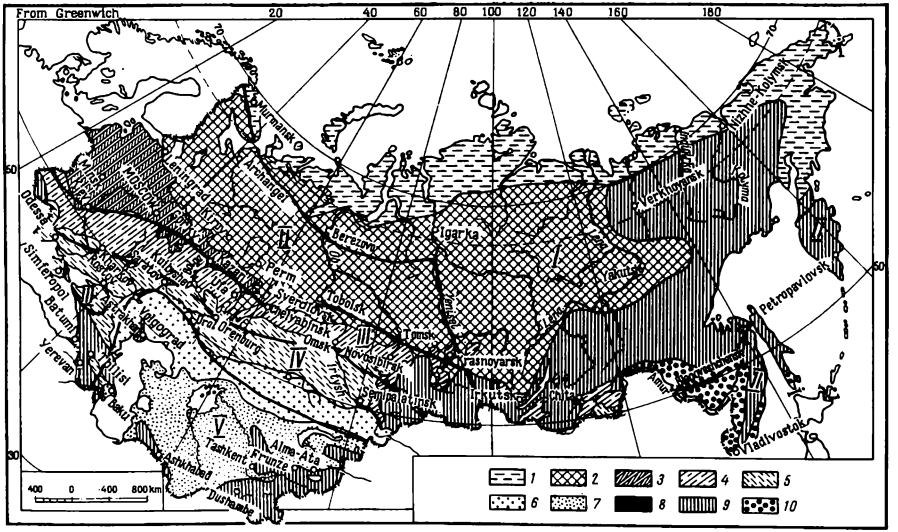


Fig. 56. Landscape demarcation of the U.S.S.R. according to Academician L. S. Berg, and road zone demarcation of the Asiatic part of the U.S.S.R.:

1—tundra; *2*—taiga; *3*—mixed forests; *4*—forest-steppe; *5*—steppe; *6*—semi-desert; *7*—desert; *8*—subtropics; *9*—mountainous regions; *10*—broad-leaved and mixed forests

balance is close to unity. In certain years the inflow of moisture may exceed the outflow, and on other occasions the reverse may happen.

Zone IV is the zone of inadequate moisture content and comprises vast chernozem territories. In its northern part the zone corresponds to that of forest-steppe, on the right bank of the Dnieper River mixed forests occur and the southern part of the left bank comprises the steppe. In this zone moderate moistening of the upper layers of soil occurs because of an appreciable evaporation and a reduced rainfall. The coefficient of water balance is 0.5-0.6. The ground water occurs only at a considerable depth.

Zone V is the arid zone. Within this zone the hydration of the soil is insignificant on account of the high rate of evaporation. The boundary of Zone V is approximately coincident with the landscape zone of the dry steppe and of the semi-desert. Zone V is characterized by the brown and chestnut soils partly including alkaline and saline soils.

The conditions for freezing of the ground are more favourable in the western part of the above climatic zones than in the east, where the climate is more continental, the rainfall is less and consequently, the speed of frost penetration is higher. The boundary between the western and eastern parts of the zone is assumed to be along the rivers Northern Dvina and Volga.

The highlands of the Caucasus and of Central Asia are not included within the road demarcation zones. The vertical zonality of soils in these regions, the rocky and stony ground, as well as the pronounced dependence of moisture conditions on the height above sea level and on the orientation of slopes in relation to cardinal points, do not permit the soil classification of these regions for road engineering. The same applies to the coastal areas where, independent of their geographical position, the conditions of soil moistening will have to be determined for each specific case.

30. Estimation of Hydrologic and Hydrogeological Conditions

For determining the elevation difference and the design thickness of the road pavement it is vital that an assessment be made of hydrologic and hydrogeological conditions prevailing, since these conditions will influence the variation of moisture content of the roadbed.

The road zone demarcation gives only the general characteristics of the specific region. The location selected for the road may substantially affect the soil characteristics of the road foundation. To take care of this contingency there has been introduced the concept of the hydrologic grouping of regions within the country.

By means of rational design it is always possible to minimize the effect of unfavourable combinations of hydrologic and hydrogeological factors on a road. However, this may entail an appreciable increase in the construction cost and has to be justified by engineering and economic reasons, which must be measured against the possibility of bypassing the unfavourable districts.

The areas of route location may be divided into three types according to the local inherent moisture content of the soil.

1. *Dry places* with moderate moisture content. These include areas with assured surface runoff, without tendencies for bog for-

mation and with the ground-water table at some considerable depth; also regions with highly permeable outcrops of extensive thickness.

2. *Damp places* with excessive moisture content for some periods of the year—places with restricted surface runoff, with excessive moistening by surface water, with a deep ground-water table and an extensive catchment, and where stagnant water collects in spring and in autumn. Also regions with symptoms of bog formation and regions where runoff is difficult owing to the soils having a low permeability (sink holes and minor depressions on wide watershed plateaux, lowlands, plains, trenches and the lower part of extensive slopes covered with forests).

3. *Wet places* with excessive moisture content, ones that are characterized by constant saturation of the upper soil strata with ground water or with surface water standing for a long time (over 20 days), peat soils, gley soils with tendencies to bog formation, saline soils and continuously irrigated areas of the arid zone.

Before relegating a route zone to one or other type according to the hydrologic conditions one should examine the topographic features of the location and its soil survey data.

The vegetation of a region often serves as a visual guide for the preliminary appraisal of the soil and hydrologic conditions, since certain groups of vegetative forms are encountered only in definite combinations of soils and hydrologic conditions.

A sharp alteration of ecology is usually associated with variation of soil particle size and of soil moisture content. The presence of ground-water discharge is shown by the appearance of moisture-loving vegetation among that preferring dry conditions, by the freshness of vegetation, succulence of foliage and the thickness of the turf covering.

In steppe and arid zones the vegetation gives a good indication of the degree of soil salinity.

In determining an acceptable route for a road, the surveyor must be fully aware of the relation between the types and manner of growth of vegetation and the corresponding hydrogeological conditions.

CHAPTER 6

ROAD DRAINAGE

31. Determination of Water Inflow Towards the Highway from the Surrounding Country

Water flows towards a road down the slopes from the surrounding country when it is raining or the snow is thawing. The assessment of the amount of water inflow is a complex problem, since the surface runoff depends on the climatic conditions, the size of the catchment area, the ground slopes, the vegetative cover, the soil conditions and a number of other factors.

The study of the surface water runoff is an art of long standing.

Storm-water runoff from small drainage areas. The modern method of estimating the runoff requires the handling of several independent variables.

1. The determination of the amount of rainfall, or water from thawing snow which is typical for a given climatic region. This is based on data derived from meteorologic station investigations over a number of years, enabling precipitation characteristics for various climatic regions to be deduced.

2. The estimation of runoff losses due to ground absorption, evaporation, water retention by vegetation and the irregularities of area surface.

3. The estimation of the influence of the magnitude, shape and slope of the catchment area on the runoff of rain and thaw water.

In the first stage of highway design the storm-water runoff can be calculated by using the simplified formula given by the Soviet scientist E. V. Boldakov

$$Q = \psi (h - z)^{3/2} F^{2/3} \beta \gamma \delta \qquad \text{m}^3/\text{sec} \qquad (106)$$

where ψ = physiographic coefficient which is related to the area topography. In low-lying areas (swamps) ψ = 0.04-0.05, in flat country—0.06-0.08, in hilly areas—0.09-0.11, in mountainous regions—0.12-0.14 and in highlands—0.15-0.16

h = "depth of runoff" which is the average thickness of the rainfall layer over the whole catchment area during the shower period, less the depth of the water absorbed by the ground

z = depth of rainfall retained by the vegetation growing in the area and by the irregularities of the ground surface. In the northern and western regions of the U.S.S.R.

many catchment areas contain swamps and lakes which retain the water and reduce the inflow towards the road.

The influence of swamping is taken into account by adjusting the assumed value of the retained depth of rainfall z. For dense grass and thin shrub its value is taken equal to 5 mm; for forest of medium density and brushwood—10 mm; for dense forest—15 mm; for taiga, chocked forests and mossy bogs—20 to 40 mm

F = catchment area, km²

γ = coefficient which takes account of the fact that the rain belt may cover only part of the catchment area. The greater the area, the greater is the likelihood of this occurring. For very long catchment areas (>35 km) it is usual to assume $\gamma = 0.8$, for medium lengths (10 to 35 km) $\gamma = 0.9$, for areas shorter than 10 km $\gamma = 1$.

The simplified formula (106) has been obtained by taking into consideration the relation between the water runoff towards the road and the extent of the catchment area, its topography and retention of rainfall in the soil and by the vegetation. The numerical values of the coefficients and the exponent of F and $(h - z)$ were obtained by statistical analysis of data from an accurate investigation.

Since in large drainage areas the water inflow following showers in remote catchment districts is retarded, a correction factor β was introduced into this formula, which takes account of the delay of runoff after the shower. The values of β are given in Table 6.

TABLE 6

Catchment area characteristics	Distance from structure to centroid of catchment area L_0, km							
	1	2	3	4	5	6	7	10
	Value of correction factor β							
Flat and hilly areas	1	0.95	0.9	0.85	0.8	0.75	0.7	0.6
Mountainous areas and highlands	1	1	1	0.95	0.9	0.85	0.8	0.7

The incidence of lakes is taken into account by the introduction of the correction factor δ for the computed flow given by Table 7.

The values of the total depth of runoff h take into consideration the absorption of rainfall by the soil and correspond to the probable frequency of a storm intensity exceeding a certain rated value. This value is chosen according to engineering considerations depending on the economic importance of a given structure for normal highway

operation, and on the costs of its erection and repair in case of damage due to excessive floods.

The design probabilities of excessive rainfall recommended for various types of structures are given in Table 8.

TABLE 7

Area covered by lakes, %	Value of δ according to the situation of lakes	
	In the higher part of the area	In the lower part of the area
2	1.0	0.9
4	0.9	0.7
6	0.8	0.5
8	0.7	0.4
10	0.6	0.3

TABLE 8

Types of structures	Rated probability of excessive rainfall for highways of class	
	I and II	III and IV
Embankments	1 : 100	1 : 50
Bridges and culverts	1 : 100	1 : 50
Drainage ditches	1 : 50	1 : 25
Tunnel entrances	1 : 1,000	1 : 1,000

Calculation according to the more accurate method takes into account the influence of rainfall duration on the depth of runoff. When using the simplified formula the rainfall duration is considered as being constant, and equal to 30 min.

Table 9 gives a range of soil coverings for runoff calculation.

The depth of runoff h corresponding to rainfall duration of 30 minutes is determined with the help of special tables, compiled for typical climatic regions, into which the territory of a country is divided. The same tables give the depths of runoff h for various climatic regions, for a variety of catchment area surface deposits and for various probabilities of exceptionally heavy and prolonged runoff. The territory of the Soviet Union is divided into 10 typical rainfall regions for this purpose.

TABLE 9

Area characteristics	Soil absorption category in relation to climatic conditions			
	Forest zone and forest-steppe	Steppe	Deserts	Monsoon climate
Nonfissured rock, asphalt and cement concrete	I	II	III	I
Clay, loamy alkali soil, takyr soil	II	III	IV	I
Loam, nontextured chernozem. Podzol soils and grey forest soils	III	IV	V	II
Textured chernozem, sandy loam alkali soils, grassed sandy loam	IV	V	VI	III
Open sandy loam	V	VI	VI	IV
Drift sands	VI	VI	VI	V

In compiling the table of the depths of runoff the following investigations have to be completed.

From the rainfall intensity for various storm durations observed at the relevant meteorological stations, rated depths of runoff are chosen according to the probability that they will be exceeded once in a given number of storms or years. From this, the appropriate rainfall intensity is found for the heaviest shower of 5 minutes duration, then of 10 minutes, 20 minutes, etc. The depths of runoff of a given probability of being exceeded are compiled by the usual statistical methods employed in hydrology. Then, upon analyzing the results obtained from self-recording rain gauges, the typical characteristics of rainfall for each examined region are determined. According to tests of water absorption by various soils, curves of water accumulation (hydrographs) against time are plotted. The methods used in these experiments are described in hydrology courses.

Locating on the same diagram the curve of rainfall of a given duration and the appropriate curve of absorption, the point of tangency C is found (Fig. 57), which corresponds to the beginning of surface runoff. The water ceases to contribute to the runoff when the rain stops. Hence the duration of runoff t equals the duration of contribution. Having found the depth of runoff as the difference between the depth of rainfall and the depth of absorption for various

rain and runoff durations, the rated depths of runoff during 30 minutes are found for various soils by interpolation, since each type of soil has its own critical storm duration.

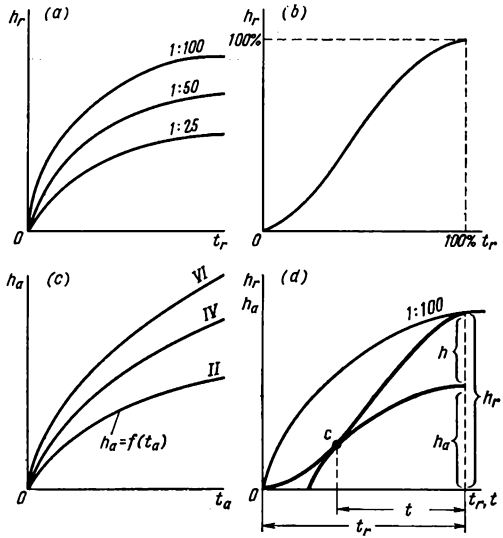


Fig. 57. Diagram for determining the depth of runoff h and the duration of runoff t

a—curves showing the layers of precipitation h_r depending on its duration t_r and the probability of its being exceeded; *b*—curve showing how rain goes on (in %); *c*—relation between depth of absorption h_a and time t_a for soils of various categories; *d*—determination of the depth of runoff, $h=h_r-h_a$, and of the runoff time t, for a given rainfall duration t_r

For very small catchment areas the simplified formula (106) gives exaggerated values. Because of this a limitation is introduced, according to which the runoff determined by formula (106) should not exceed the total quantity of rainfall per unit time on the catchment area. Therefore, when using formula (106) for calculations, it is necessary to check the results by means of the following inequality:

$$Q \leqslant 0.56hF\beta\gamma\delta \quad \text{m}^3/\text{sec} \tag{107}$$

For a precise assessment of the local conditions, for example, in the case of an important highway or structure in the working project stage, an accurate method should be employed. Such a method has been developed in the U.S.S.R. for determining the storm-water runoff let through by small bridges and culverts. It consists in plotting a hydrograph for the runoff of the design catchment area, characterizing the variation of water inflow towards the bridge or culvert in time and allowing estimation of the total amount of water discharged during the rain, as well as the maximum inflow per second towards the structure. To plot the discharge hydrograph, data from auxiliary tables are used, based on statistical analysis of meteorologic observations, and on experimental determination of the coefficient of water absorption by various soils, etc.

32. Highway Drainage

The highway drainage system comprises a series of special structures and separate devices provided for safeguarding the roadbed against saturation. Their purpose is to collect and dispose of surface and ground water flowing towards the road, and to prevent water from entering the subgrade. As a result of these measures a stable moisture content of the subgrade should be established.

To dispose of the surface water falling on the road, the following measures are necessary.

1. The cross-sections of both the formation and the pavement are given a convex profile, the crossfall providing for lateral drainage flow.
2. Side ditches and flumes are constructed, and in certain cases catch pits and evaporation reservoirs may be provided.
3. Intercepting ditches are made, collecting the water which runs down the slopes of cuttings, etc., towards the road.
4. Bridges, culverts and filtering terraces are constructed for the purpose of discharging the water from side ditches across and under the road, also measures may be taken to divert the water from the roadbed.

If water disposal is not assured and there is a possibility of ponding next to the road for a prolonged period, it is necessary to build the road on an embankment above natural ground-level or above the permanent static water level, or ground water table, etc., in order that the capillary rise should not reach the subgrade. The building-up of the formation level is the most effective method for guaranteeing the stability of the roadbed. The construction of an embankment should present no difficulty with modern mechanized techniques and constitutes a comparatively small percentage of the total road cost.

To prevent the action of ground water on the road, the road pavement sub-base can be elevated above the ground-water table (Fig. 58), membranes to prevent capillary, film and vapour moisture transfer can be placed in the roadbed (for details see Sec. 38), and drains may be laid in order to lower the ground-water table.

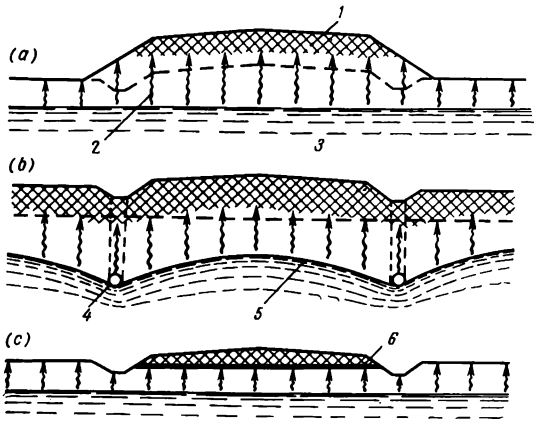


Fig. 58. Ways of countering the harmful effect of ground water on the roadbed:

a—elevation of road sub-base; *b*—laying of drains; *c*—location of a membrane; *1*—dry ground; *2*—zone of capillary rise; *3*—ground water; *4*—drain; *5*—line of saturation; *6*—impervious membrane

The system of road drainage also includes the construction of bedding (percolation) courses of sand, gravel and other coarse-grain material, where the ground water percolating from verges, fissures and pavement joints is collected. From the percolation course the water is diverted on to the embankment slopes or into side ditches by means of special drain outlets. In spring the percolating course absorbs and retains water percolating from the upper layers of the roadbed upon the thawing of the ice covering formed on frost heave sections during the winter accumulation of moisture. Sand blankets are used in the 2nd and 3rd climatic zones in case of excessive and variable moisture content and when the roadbed is erected from cohesive and loamy, silty and loamy, and silty and sandy loam soils.

The drainage layers are computed for the condition of absorbing all the water flowing into the base course of the carriageway.

Depending on the width of the carriageway and the climatic region, the compacted sand materials for the percolating membrane should have a filtration coefficient from 3 to 10 m/day.

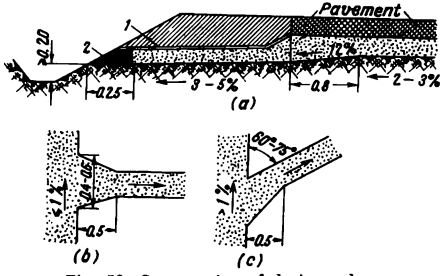


Fig. 59. Construction of drain outlets:

a—longitudinal section of drain outlet; *b*—junction of the outlet to the roadway with small longitudinal gradients; *c*—ditto, with steep longitudinal gradients; *1*—layers of inverted turf; *2*—filling the end of the outlet with rubble or gravel

The depth of the sand membrane for various types of surfacing should be as indicated in Table 10.

TABLE 10

Type of pavement	Nature of subgrade	Thickness of sand layer (in centimetres) for the following moisture conditions in subgrade		
		Saturated	Normal	Low
Cement concrete	Fine sand	15	10	10
	Fine sandy loam	20-25	15-20	10
	Heavy loam and clay	25-35	20-25	15
	Powdery soil and loam	35-50	25-40	15-20
Flexible, for class I-III roads	Fine sand	10	—	—
	Fine sandy loam	20	15	10
	Heavy loam and clay	30	20	15
	Powdery soil and loam	35	25	20
Flexible, for class IV-V roads	Fine sand	10	10	0
	Fine sandy loam	15	15	10
	Heavy loam and clay	25	20	15
	Powdery soil and loam	30	20	15

The drainage layers, as a rule, are designed over the whole width of the roadbed. In some cases, however, they are designed over the width of the carriageway, and drains are provided for the removal of water from these layers. The drains are filled with permeable material—uniform rubble, pebbles of 40 to 60 mm size, etc., through which the water from the roadbed may percolate.

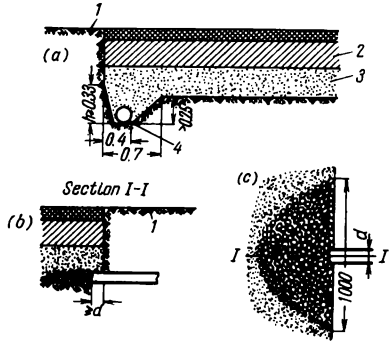


Fig. 60. Drains for draining a sand base:
a—longitudinal land drain; b—inlet of a spur drain; c—plan view of pipe drain inlet; 1—shoulder; 2—pavement; 3—sand base; 4—longitudinal drain

The drain outlets are of 0.4 × 0.2 m cross-section and are staggered with a spacing of 4 to 6 m (Fig. 59).

The capacity of the individual drain outlet is not large, and therefore appreciable time is required for the disposal of water accumulated in the pores of the sand base. Because of this, in places with unfavourable soil conditions the water from the bedding course is discharged by means of lateral and longitudinal asbesto-cement and ceramic drain pipes (Fig. 60).

33. Road Pavement Camber

To facilitate the disposal of water from the surface the carriageway is given a camber sloping away from its centre line and towards its shoulders. The less the evenness of the surfacing, the greater should be the lateral gradient, since the water may be retained in the recesses which obstruct its flow and percolate inside the pavement. However, to suit the traffic requirements the lateral gradient has to be reduced to the minimum necessary for the efficient

disposal of water. The necessity to reduce the lateral gradient is demonstrated by the following considerations:

1. With considerable lateral gradients and with a slippery road surface the vehicles may slide on the surfacing; this effect is especially noticeable on smooth cambered earth roads after a shower, when the road is covered by a thin layer of mud.

2. When vehicles having double wheels draw out into the middle of the carriageway, their inner wheels are overloaded (Fig. 61) causing excessive wear of tyres and of the surfacing.

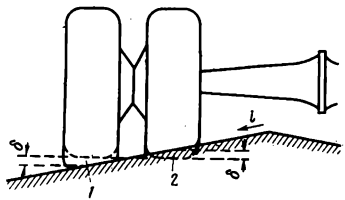


Fig. 61. Overloading of inner wheels when double-wheel vehicle draws over to the middle of carriageway:

1—reduction of tyre compression compared with running on a horizontal surface; *2*—additional compression of tyre due to overloading of the inner wheel

3. The lateral gradient of the surfacing encourages tyre side slip, rendering steering more difficult, and also rapidly wears out the tyres.

The lateral gradients adopted in relation to the type of surfacing are given in Table 11.

TABLE 11

Type of pavement	Surfacing lateral gradient, per cent	
	Minimum	Maximum
Asphalt concrete and cement concrete	15	20
Gravel and crushed stone stabilized with organic binders, mosaic, stone block and clinker pavements	20	25
Gravel and crushed stone	25	30
Pavements of cobblestone or broken stone, soil pavements stabilized with local materials	30	30

The shoulders receive a more pronounced slope than the main carriageway, because their surfaces may become churned up by vehicles which pull up there in unfavourable weather, whilst the ponding of water on the ground usually causes saturation of the road sub-base. Depending on the type of soil of the roadbed and on the type of surfacing, the shoulders are given a crossfall 1 or 2% greater than that of the pavement.

The carriageway camber is usually parabolic, or consists of two straight slopes joined at the centre by a circular strip 2 m wide. With the parabolic camber the transverse gradient is specified as the average between the most convex part of the profile and the shoulders.

The shoulders of class I-III roads are levelled, compacted and sown with short-stemmed grasses in order to create a turf covering.

Within populated areas, where frequent pulling up of vehicles on the road shoulders is likely, the surface of the shoulders is stabilized with gravel, chippings, slag, locally obtained rubble, or treated with binding agents. If the shoulders are not stabilized and there are no kerbs along the pavement edges, the latter may become rutted and disintegrate.

34. Ditches

For collection of water from the roadbed, side ditches, flumes, interception ditches and drain channelling are used.

Side ditches in cuttings and next to embankments are excavated to a depth of up to 0.6 m. These ditches are for the collection of water flowing off the road surface and from the adjoining land during rainfall or snow thawing. The side ditches contribute to the drainage of the subgrade because of the evaporation of moisture from the side ditch inner slopes. However, the major use of the side ditches is to permit rapid discharge of water. When this water discharge is not ensured and ponding occurs, the ditches become a source from which water may penetrate back under the road, resulting in saturation of the subgrade.

In the case of impermeable soils and in less favourable conditions of runoff the side ditches are given a trapezoidal cross-section (Fig. 62*a*) with a bottom width of 0.4 m and a depth of up to 0.7-0.8 m from the edge of the embankment. The cuttings are battered to a slope of 1:1.5, while for embankments the inner slope is 1:3.

If the road is built in dry country with a rapid surface runoff, and the occurrence of ground water is deep, the side ditches are given the shape of triangular flumes of 0.3 m minimum depth (Fig. 62*b*). The steepness of the flume slope is limited to 1:3 which permits vehicles to draw off the road in an emergency.

In permeable sandy, gritty and gravelly soils, where the absorption of water is rapid at any time of the year, ditches are not considered necessary.

In cuttings sited in gravelly, gritty, or in soft, easily weathered, rocky grounds, trapezoidal ditches are built with a minimum depth of 0.3 m, the slopes being battered at 1:1. In solid rocky ground triangular ditches are made with a minimum depth of 0.3 m, the inner slope being trimmed to a pitch of 1:3 and the outer one of 1:1 to 1:0.5, depending on the type of ground.

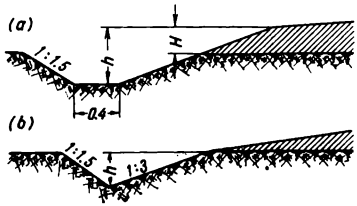


Fig. 62. Cross-sections of roadside ditches:
a—trapezoidal; *b*—triangular

The depth of the ditches in flat country is determined according to past operational experience within the above range, the capacity of ditches being checked where necessary for runoff from adjoining slopes by means of hydraulic computation. The probability of flooding must be related to the class of road. The depth of the ditches must be such that the discharge apron of the drain is at least 20 cm above the ditch invert.

When designing the roadbed the following frequency of occurrence for the maximum height of the ground-water table is taken:

Road class	I	II-III	IV-V
Frequency in years	100	50	33

To discharge water rapidly, the side ditches are given a longitudinal fall which should not be less than 0.5% in road zones I-III, and 0.3% in zones IV and V. If these conditions cannot be satisfied, the embankment elevation difference must be raised sufficiently for the pavement sub-base to be above the level of permanent ground-water saturation. Along stretches of road with steep longitudinal gradients the side ditches are designed according to hydraulic calculation from considerations of the amount of water flowing into the separate ditch sections from the road and the adjacent land. The calculation for each separate section must take into account the accumulated increase of flow.

The water from the side ditches should be discharged away from the road on the downgrade side at intervals of not more than 500 m. If the road is laid on a hillside, water from an interception ditch should be diverted under the road with the aid of *culverts*. Where the road changes from cutting to embankment the ditches on the uphill side must connect into a borrow pit, and from the downgrade they must be led out on the surface of the ground away from the cutting (Fig. 63). To facilitate the flow of water along the borrow pit its floor is thoroughly levelled after the termination of earthworks and given a gradient of 2% falling away from the embankment. Borrow pits having a width exceeding 6 m are given a concave profile falling towards the centre. If the borrow pit has a longitudinal gradient of less than 0.5% a ditch 0.4 m wide is dug in the middle in order to improve the discharge of water. To prevent soil erosion by water all ditches diverting water from borrow pits should be thoroughly stabilized.

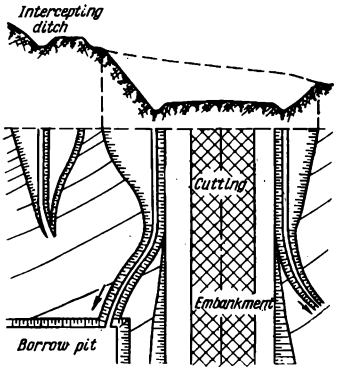


Fig. 63. Discharge of water from intercepting and side ditches in cuttings

Flumes (*drainage ditches*) are provided for the purpose of discharging water from the side ditches or catch pits situated next to the road into nearby valleys and depressions. The cross-section of the flumes is usually made equal to the cross-section of the ditches from which the water is diverted.

To improve water discharge by the flumes and to reduce the volume of work required for their construction, the side slopes should be made as steep as ground stability allows.

In order to avoid erosion and flooding the flumes are made to coincide with the natural watercourses and following an easy curve with a minimum radius equal to ten times the upper width of the ditch.

Intercepting ditches are used for the collection of water flowing down the hillside towards the road, and for diverting it into the nearest drains, borrow pits and land depressions.

The intercepting ditches are made with a trapezoidal cross-section, the dimensions of which are determined by hydraulic calcula-

tions. These computations must take into account the progressive increase of the ditch catchment area with an increase in the distance from the watershed.

For this reason cross-sections of the intercepting ditches are usually calculated for individual portions according to the increase of the catchment area. As an alternative to an intercepting ditch two parallel ditches having a smaller cross-section can be constructed.

The intercepting ditches must be cut at a longitudinal gradient less than that which would result in scouring of the channel.

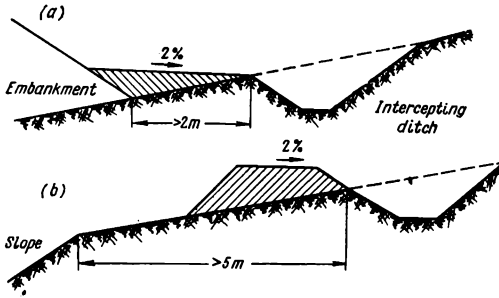


Fig. 64. Arrangement of intercepting ditches:
a—next to an embankment; b—next to a cutting

To avoid wash-outs and sliding of slopes in a cutting where seepage occurs or which could occur as a result of accidental silting up of intercepting ditches, the distance of the latter from the edge of the cutting should be at least 5 m. On hillsides with a gradient less than 1:5 the soil from the intercepting ditch should be used to build a low *ridge* (*bank*) between the cutting and the intercepting ditch (Fig. 64). This bank protects the road from being flooded on occasions when the intercepting ditch is overtopped.

Drain ditches are used for lowering the ground-water table at points where it is close to the ground surface. Usually drain ditches are built instead of pipe subdrains when the road is laid in marshy ground, in which case the discharge of water from the roadside becomes part of the main drainage for the adjacent area.

The ground-water table between the ditches will be established along a curve of depression and this is also true for subdrains (see Sec. 39).

Water will flow along side, drainage and intercepting ditches with a velocity dependent on the longitudinal gradient, cross-section of the ditch, depth of the stream and degree of roughness of slopes and base. When the flow velocity is less than the self-cleansing velocity of 0.4-0.5 m/sec the soil particles suspended in the

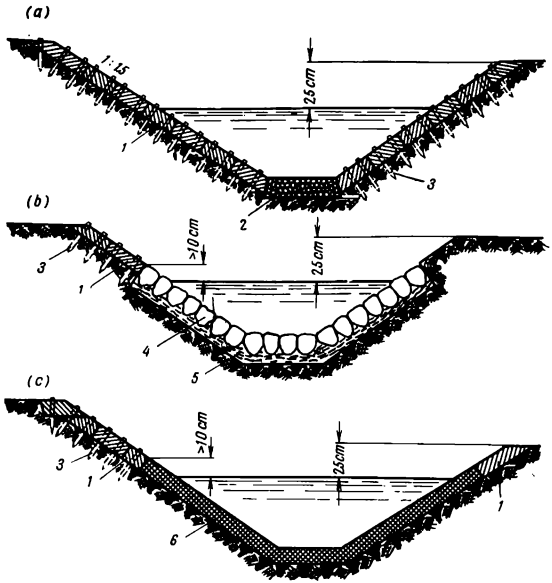


Fig. 65. Strengthening of side ditches:

a—continuous turf strip; *b*—paving; *c*—soil stabilized with organic binder; *1*—turf 8-10 cm; *2*—rubble or gravel layer 8-10 cm; *3*—wooden pegs 2.5 × 2.5 cm in section and 25-30 cm long; *4*—stone 12-16 cm; *5*—layer of moss, hay, or straw, 3-6 cm; *6*—soil stabilized with binders, 5-10 cm

water settle gradually to the bottom, causing silting. This causes blockages with resultant ponding. The minimum self-cleansing gradient for drainage ditches is 1 in 500.

When the flow velocity is too great the soil of the ditch becomes eroded, hence ditches should be properly strengthened to obviate this. The side slopes of ditches should be turfed to a height 0.1 m

above the design water level and their floors lined with rubble, back-filling stabilized with organic binding agents, or a single coat of paving and cement-rubble masonry (Fig. 65). The degree of reinforcement of drainage ditches should be determined according to hydraulic computation in relation to the velocities given in Table 13 (page 151).

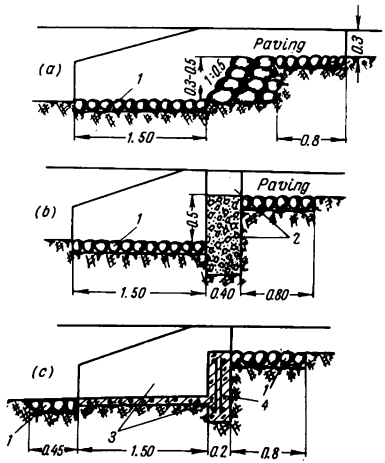


Fig. 66. Design of weirs:

a—rubble masonry; *b*—concrete or rubble concrete; *c*—precast reinforced concrete members; *1*—paving; *2*—concrete or rubble concrete; *3*—reinforced concrete members; *4*—reinforcement bars

For side ditches the cross-section of which has not been determined in accordance with hydraulic computation, the type of protection is chosen depending on the magnitude of their longitudinal gradients and with a view to previous experience (Table 12).

With steep gradients the bottom of the ditch is given a stepped profile, i.e., cascade ditch weirs are constructed of precast reinforced concrete units, of in-situ concrete or of rubble; for roads of local importance these may be of wattle with gravel in-filling (Fig. 66).

The portions of the ditch bottom adjacent to ditch weirs should be reinforced with paving. Between the weirs, the ditch may be given a simple fall, where compaction is not required or which corresponds to the accepted type of reinforcement.

TABLE 12

Nature of bank protection or construction	Longitudinal gradient of ditch (%)	
	Sandy soil	Clayey soil
Unprotected	<1	<2
Turfed banks	1-3	2-3
Slab pavings	3-5	3-5
Weirs and flumes	>5	>5

The distance between the weirs is determined from the equation

$$L=\frac{h}{i_2-i_1}$$

where h = height of the weir
i_2 = road gradient
i_1 = gradient of the ditch between the weirs.

It is worthy of note that the majority of types of protection in present use are of an appreciably labour-consuming nature and have to be executed by hand. The problem confronting investigators is that of evolving such types of protection as lend themselves to mechanized construction. Soil stabilization seems to offer great possibilities, viz., silty clayey loessial soil, treated with 10% of bitumen, will withstand water velocities of 2 to 3 m/sec.

35. Evaporation Reservoirs

In arid steppe regions, and in flat country when it is not possible to discharge water by means of side and drainage ditches into natural land depressions, *evaporation reservoirs* are constructed at the side of the road. These reservoirs are excavations around which earth ridges are made in order to prevent the inflow of water from the surrounding country. Sometimes, instead of special evaporation reservoirs borrow pits are used, which, in this case, are situated at further distances apart than usual.

The capacity of a single evaporation reservoir must not exceed 200-300 m^3, its depth must not exceed 1.5 m, and the water level should be 0.6 m lower than the elevation of the roadbed edge. The design of the evaporation reservoirs consists in choosing such a capacity as would enable the amount of water flowing from the roadbed during a rainfall to be fully evaporated between successive showers.

The averages needed for design purposes are obtained from the meteorological stations.

The evaporation capacity is determined according to the formula

$$e = 11.5d\,(1 + \alpha v) \tag{108}$$

where e = evaporation, mm per month
d = air humidity deficit, mm of mercury column
v = wind velocity, m/sec
α = coefficient depending on the height above ground level at which the wind velocity is measured, and which is taken as follows:

Height H, m	0.3	1.0	9.0
Coefficient α	0.43	0.27	0.14

The air humidity deficit can be determined according to the approximate formula

$$d = i - w \tag{109}$$

where i = maximum water vapour pressure at the average monthly temperature, mm
w = monthly average absolute humidity, mm.

The values of i and w are obtained from climatic records or from data supplied by meteorological stations.

Thus, the required area of the evaporation reservoir in square metres can be determined according to the formula

$$\omega = \frac{0.001F\,(a - e)\,\psi}{h} \tag{110}$$

where F = catchment area for a single evaporation reservoir, m²
a = quantity of rainfall during the most rainy summer month according to records for many years, mm
h = depth of the reservoir, m
ψ = runoff coefficient, making allowance for the absorbed rainfall, which may be assumed—depending on the soil conditions—to have a value between 0.3 and 0.5.

The dimensions of the reservoir can be calculated more accurately according to the total annual water accumulation, provided that at the end of the summer the reservoir should be dry. In this case one has to calculate for each month the amount of water which remains in the evaporation reservoir.

Evaporation reservoirs should be built only in localities where the climatic conditions (restricted rainfall, high average annual air temperature and high winds) encourage high evaporation. In regions with a humid climate and impervious soil they will only further the saturation of the ground.

Evaporation reservoirs require an additional allocation of land for them. If they are not subject to daily maintenance, the reser-

voirs will become overgrown with weeds and be sources of pollution for the neighbouring agricultural fields. This is why on stretches with a difficult water discharge it is advisable to design the roads on embankments with such an elevation of the roadbed that is sufficient to make the arrangement of ditches and evaporation reservoirs unnecessary.

36. Structures for Water Discharge

Provision for the passage of running water beneath the road may be necessary where the highway crosses rivers, ravines or gorges, where surface water is discharged after rainfalls or the thawing of snow. The nature and number of structures will depend on the climatic conditions and the topography, and the cost of these structures may be as much as 8 to 15 per cent of the total cost of a modern motor highway. Therefore, the correct choice of the type of structure and the rationalization of their design to permit mass production of the components, can appreciably reduce the cost of their construction.

In various climatic and topographic conditions the average number of structures (bridges, culverts, etc.) per km of road is approximately as follows:

	Average number of structures required per 1 km of road
Deserts and semi-deserts	0.3
Marshy ground	1.0
Flat country	0.5-1.0
Moderately undulating country	0.7-1.2
Very rough country	1.0-1.5
Mountain regions	1.5-2.0
Regions with artificial irrigation	3.0

The main structures for allowing the passage of water are *bridges* and *culverts*. Other means are used occasionally to divert water either directly through the road foundation—*filter banks*—or by flowing over the road—*gutters*.

Structures for diverting water have to be convenient for the water flow and permit its discharge without damage to the road structure. These requirements, together with making the structures economical, result in complex problems which have to be solved by comparison of alternative designs.

The majority of water discharging structures built on highways are small bridges and culverts. From the point of view of motor transport requirements, the best structure for any particular case is the one which does not alter the conditions for traffic, does not

require horizontal or vertical diversion of the route alignment, does not hinder the construction of the carriageway or the shoulders, or require alteration of the type of pavement. From this aspect, the best type of small-capacity water discharge structures are culverts, which can be laid easily with any combination of road profile and plan, and with high embankments, using the same type of pavement for the whole length of the road. Therefore, culverts constitute almost 85% of the total number of water channeling structures on highways.

The building of bridges makes high demands on the road profile. The setting of bridges on vertical or horizontal curves, or following extensive longitudinal gradients, though constituting one of the basic elements of highway location, nevertheless complicates their construction. Often one has to use a different type of pavement on bridges than on their approaches; the great height of the embankment for crossing deep ravines makes it necessary to build bridges of considerable length, despite a low flow of water, and this increases very considerably the cost of construction.

All the above considerations indicate that culverts are the main type of small water channeling structures for water courses having a continuous or occasional flow of up to 10 m^3/sec and free from floating ice. In modern road building most bridges and culverts are constructed of reinforced concrete and are often assembled from prefabricated components. In mountain regions on roads of lower classes the culverts may be constructed of dry masonry.

To increase the capacity of the culverts without increasing the height of the embankment, multiple culverts are made by laying several tubes parallel to each other. Investigations show that in these cases the flow is equally distributed between the tubes.

Bridges are classified into three categories according to their effective span between abutments: *minor* bridges having spans of up to 30 m, *medium* ones with spans of less than 100 m, and *major* ones with spans of over 100 m.

To facilitate their design and erection, the dimensions of minor bridges and culverts are usually standardized, and have to be adapted to individual cases.

In the U.S.S.R. the standard internal diameters of round culverts are: 0.75; 1; 1.25; 1.5 and 2 m; the openings of rectangular, ovoid and arched culverts are: 1; 1.25; 1.5; 2; 2.5; 3; 4; 5 and 6 m; while spans of small-span standard reinforced concrete bridges are: 1; 2; 3; 4; 5; 5.5; 6; 7.5; 9 and 12 m.

Filtering embankments of uniform rubble (Fig. 67) are built only when frost-resistant stone is available close to the site. If the stone has to be transported more than 2 to 3 km it is not worth building a filtering bank since the quantity of stone required for ensuring

a water-filtration capacity of 1 m^3 through the embankment may be as much as 80 to 100 m^3.

Filtering banks can be built at road crossings where continuous or intermittent water courses occur, provided that the catchment area and the bed are of stable soil, not susceptible to scour.

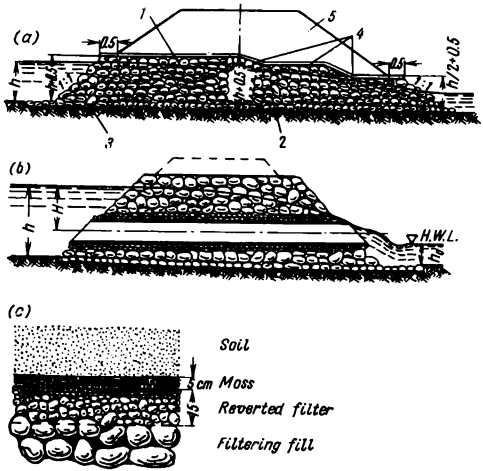


Fig. 67. Filtering banks:

a—continuous; *b*—with pipe laid to increase discharge capacity; *c*—detail of rubble fill and soil bank connection; *1*—saturation curve at the highest possible water table; *2*—filtering bank; *3*—paving of the bed; *4*—isolating membrane (bituminous soil, moss, straw); *5*—roadbed

Otherwise, at reduced flows with consequent low velocity and small head, the interstices of the rubble filling may become silted up.

When the water behind the filter bank backs up to a height in excess of 0.5-0.7 of the embankment height the water flow velocity through the filter bank may cause erosion of the earth foundation beneath the embankment.

To increase the capacity of the filter banks round culverts can be laid within the rubble filling; these serve as penstocks.

Gutters with paved surfacings can be used on roads of lower classes when they are crossed by intermittent water courses having maximum depths of 0.15-0.20 m.

37. Calculation of Water Channeling Structure Openings and River Bed Protection

Minor bridge openings are calculated according to formulas of hydraulics for water discharge over broad crested weirs. Depending upon the water level in the water course below the bridge two cases are possible (Fig. 68):

1. Free flow, when the level of the water does not influence the depth of the water under the bridge, which corresponds to the average critical depth

$$h_{k.\,av} = \frac{v_b^2}{g} \qquad (111)$$

where v_b is the stream flow velocity under the bridge, which is assumed equal to the one accepted for a natural or a stabilized bed under the bridge, m/sec.

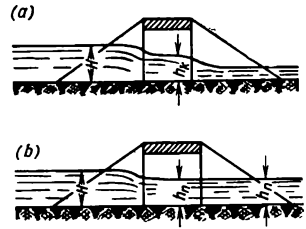


Fig. 68. Diagram of water flow through structures:
a—free flow; b—constricted flow

2. Constricted flow, when the water level under the bridge is determined by the level downstream of the bridge. This case occurs only when the natural depth is

$$h_n > 1.3 h_{k.\,av} \qquad (112)$$

Calculation according to the free flow method gives the required width of the stream along the free surface under the bridge

$$B = \frac{gQ}{\varepsilon v_b^3} + Nd \qquad (113)$$

where Q = design flow, m³/sec
N = number of intermediary supports
d = width of a support, m
ε = coefficient of contraction, depending on the form of the bridge piers and the design of the abutments
g = gravitational acceleration.

The depth of the water upstream of the bridge in this case is

$$H = h_k + \frac{h_{k.av}}{2\varphi^2} - \frac{v_u^2}{2g} \qquad (114)$$

where v_u = flow velocity upstream of the bridge (approach velocity), considered only when it exceeds 1 m/sec

φ = velocity coefficient to allow for energy losses

h_k = full critical depth, m (with rectangular and wide trapezoidal river beds $h_k = h_{k.\,av}$).

The values of the coefficients of contraction ε and of velocity φ in relation to the form of the abutments are given as follows:

	ε	φ
Buried abutments with cones	0.90	0.90
Wing abutments	0.85	0.90
Abutments protruding from cones	0.80	0.85

For calculations according to the condition for constricted outflow the depth of the stream under the bridge is assumed to be equal to the natural depth of the stream h_n. In this case the bridge opening is

$$B = \frac{Q}{\varepsilon h_n v_b} \tag{115}$$

The depth of the stream above the bridge is determined by using formula (114), assuming $h_k = h_n$ and with $h_{k.\,av}$ determined according to formula (111). The computed value of B is the width of the stream at the level $\frac{h_n}{2}$.

The natural depth of the stream is determined according to the flow Q, and the shape and slope of the waterway cross-section. For this calculation the formula of uniform flow is used

$$Q = AC\sqrt{hI} = (AC\sqrt{h})\sqrt{I} \tag{116}$$

where C = Chezy coefficient

$AC\sqrt{h} = mAh^{2/3} = f(h)$ = flow coefficient of the water cross-section

m = coefficient allowing for bed roughness

A = cross-sectional area.

The openings of round culverts (and culverts of other shapes) are calculated according to formulas of hydraulics taking into account the condition of flow in the culvert, i.e., free flowing, running full bore or surcharged. To facilitate practical calculations the design organizations have compiled tables for determining the discharge capacity of standard culverts.

Often as a result of the construction of embankments, bridge approaches, etc., across ravines and gently sloping gorges an artificial pond is formed on the upstream side of the road. This occurs because the inflowing water is partially obstructed by the embankment, and the discharge through the structure is sometimes less than the rate of inflow of water. After storm water runoff the water gradually passes through the opening and the pond is emptied.

The estimation of the potential reservoir capacity upstream of structures will often allow a considerable reduction in discharge capacity and hence in size.

According to E. V. Boldakov the minimum permissible flow through a structure is

$$Q_{min} = Q\left(1 - \frac{W_p}{W}\right) \tag{117}$$

where Q = maximum flow of area runoff, m^3/sec

W = total volume of runoff, the amount of water falling on the catchment area during a rain can be calculated, in m^3, according to the formula

$$W = (h - z)F\gamma \tag{118}$$

W_p = volume of the pond, which is determined on a plan with contour lines or according to the simplified formula assuming a parabolic shape for the cross-section at the outfall

$$W_p = \frac{22bH^2}{i}\ m^3 \tag{119}$$

where H = depth of the stream from the invert of the structure, m

b = width of the stream with the depth H, along the route, m

i = average bed gradient, expressed as a percentage, within the limits of the pond.

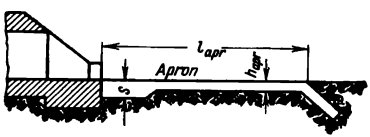


Fig. 69. Arrangement of apron downstream of structure

Water discharging from structures with an excessive velocity may erode the bed, and undermine and endanger the structure. To prevent this happening the beds beneath bridges and below the outlet from culverts may require some form of protection.

At the outlet from a structure the water flows out at an angle of 90 to 100°, and the bed must be correspondingly protected. This protection usually takes the form of a masonry or concrete apron, of a length 3 or 4 times the width of the stream at the outlet (Fig. 69).

The design of the apron provides for a maximum thickness at the outlet from the structure, where the stream has the greatest energy.

The type of apron is chosen in accordance with the outlet velocity

$$v_{apr} = 1.5 v_{out} = 0.9 \sqrt{2gH} \tag{120}$$

where H is the depth of the flow at the entrance to the structure.

The thickness of the apron is determined according to normal design considerations.

The depth of bed erosion below the apron can be determined by two factors: erosion with the formation of ridges when the stream deepens by an amount Δ_1 and erosion when the bottom current velocity exceeds the ground resistance to erosion Δ_2. The depth of erosion Δ_1 is

$$\Delta_1 = H \left[1.90 \sqrt{\frac{B_0}{B_0 + 2l_{apr}}} - 0.35 \frac{B_0}{B_0 + 2l_{apr}} \right] = Hf\left(\frac{l_{apr}}{B_0}\right) \tag{121}$$

The values of $\frac{\Delta_1}{H} = f\left(\frac{l_{apr}}{B_0}\right)$ are given in the following table:

$\frac{l_{apr}}{B_0}$	0	1	2	3	4	5	6	7	8	9	10
$\frac{\Delta_1}{H}$	1.55	0.98	0.78	0.65	0.59	0.54	0.50	0.47	0.45	0.42	0.40

The depth of erosion Δ_2 is obtained from the equation

$$\Delta_2 = 0.35 \frac{B_0}{B_0 + 2l_{apr}} \left[\frac{1.25}{\sqrt{\frac{v^2_{max.\,p}}{gH}}} - 1 \right] H = H \frac{B_0}{B_0 + 2l_{apr}} f\left(\frac{v^2_{max.\,p}}{gH}\right) \tag{122}$$

where $v_{max.\,p}$ is the maximum permissible velocity of the stream in metres per sec.

The values of $f\left(\frac{v^2_{max.\,p}}{gH}\right)$ are given below

$\frac{v^2_{max.p}}{gH}$	0	0.001	0.005	0.01	0.05	0.10	0.20	0.30	0.40	0.50	0.60
$f\left(\frac{v^2_{max.p}}{gH}\right)$	∞	13.1	5.72	3.95	1.57	1.01	0.61	0.44	0.33	0.26	0.21

The protective toe at the end of the apron is let into the ground to a depth equal to the larger of the two values

$$h_{toe} = 1.5\Delta_1 \quad \text{or} \quad 1.5\Delta_2 \tag{123}$$

The permissible velocities in natural river beds and with protection of various types are given in Table 13.

TABLE 13

Soil or type of apron	Permissible velocity, m/sec, at stream depth			
	0.4 m	1.0 m	2.0 m	3.0 m
Dust or silt	0.15-0.2	0.2-0.3	0.25-0.4	0.3-0.45
Sand	0.2 -0.65	0.3-0.75	0.4-0.8	0.45-0.90
Gravel	0.5 -1.1	0.6-1.2	0.7-1.35	0.75-1.50
Cobblestone	2 -3.5	2.4-3.8	2.75-4.3	3.10-4.65
Clay and loam	0.35-1.0	0.4-1.2	0.45-1.4	0.5-1.50
Flat turf	0.9	1.2	1.3	1.4
Grassed slopes	1.5	1.8	2.0	2.2
Single paving over moss	2.0-3.0	2.5-3.5	3.0-4.0	3.5-4.5
Ditto, over rubble	2.5-3.5	3.0-4.0	3.5-4.5	4.0-5.0
Ditto, with selection of facing and rough fixing on rubble	3.5-4.0	4.5-5.0	5.0-6.0	5.5-6.0
Rubble masonry of low-strength rock	3.0	3.5	4.0	4.5
Rubble masonry of high-strength rock	6.5	8.0	10.0	12.0
Concrete	5.0-6.5	6.0-8.0	7.0-9.0	7.5-10.0
Concrete invert with smooth surface	10.0-13.0	12.0-16.0	13.0-19.0	15.0-20.0
Wooden invert	8.0	10.0	12.0	14.0

The greater velocity values relate to coarser or highly compacted soils, to larger stone rubble or to concrete of higher quality.

For calculating bridge openings over small or medium rivers the above method of calculation is not applicable. The main difference between these bridges and small structures is the soil erosion occurring under the former at periods of the design maximum flow, while the flow under a minor bridge or in a culvert takes place in an erosion-proof, protected bed. The calculation of scouring which determines the required depth of the foundation for bridge piers is thus the most important criterion for determining bridge spans over continually flowing rivers.

The water discharging through small and medium water courses having large catchment areas, often originates from the thawing of snow or from rainfall incident only on a portion of the total area.

The main factors for the calculation of openings are the high water level during floods and the characteristics of the cross-section of the bed at the bridge point, the conditions of water flow in the bed and on flood plains, the gradients of the river and flood plain and the types of surface deposits at the crossing.

The most difficult operation is to determine accurately the high water level for design purposes. Depending on the general weather conditions, the flood level varies considerably in separate years. When determining bridge spans over big rivers one usually uses data obtained from observations at gauge stations extending over a period of years. By the use of statistical techniques, the peak flood level occurring on an average once in a sufficiently large number of years may be established. The more important the structure, the greater should be the period between occurrences of the design flood.

For less important bridges on local highways and on approach roads to building sites, and also for small and medium rivers on which gauge stations do not exist, the highest water level observed during the last few decades is usually adequate. To find out these levels one has to consult the local "oldest inhabitants" and look for traces of high water marks. For small and medium rivers a design water level during floods should be computed, taking an additional margin for safety over the one established as described above.

When building a bridge, the embankment crossing the flood plain constrains the waterway cross-section available at flood time and accordingly the water velocity increases. Under the bridge erosion of the bed may take place, increasing in direct proportion to the increase of the water velocity. The permissible constriction of the waterway by the bridge is determined by the allowable magnitude of erosion—this depends on the geological conditions of the bed and on the type of bridge piers selected.

The depth of the river bed beneath the bridge after erosion is determined by means of O. V. Andreev's formula which he derived from considerations of the equilibrium of sediments. Hence for cases where the erosion is not limited by geological conditions

$$h_e = h_{en} \frac{Q_e}{Q_{en}} \left(\frac{B_{en}}{B_e} \right)^{3/4} \tag{124}$$

Here the subscript "e" denotes the width B, the depth h and the flow Q in the river bed under the bridge after erosion, and the subscript "en" indicates the same elements in their natural state before the bridge was constructed.

To reduce the bridge opening it is recommended that the banks of the river be set back, i.e., the bed under the bridge be widened to the dimensions of its opening L. Allowing for piers situated along the bridge span, which take up a certain part of its length λL—causing

the contraction of the effective waterway between the piers allowed for by the factor μ—the minimum required bridge opening, if $Be = [L\mu(1+\lambda)]$, can be expressed by

$$L = \frac{B_{en}}{\mu(1-\lambda)}\left(\frac{Q}{Q_{en}}\right)^{4/3}\left(\frac{h_{en}}{h_e}\right)^{4/3} \tag{125}$$

The contraction factor μ for flow between piers with spans under 10 m is 0.85-0.95; the constraint factor for flow past the piers, is

$$\lambda = \frac{l_{pier}}{l} \tag{126}$$

where l_{pier} = pier width, m
l = distance between the pier centre lines, m.

The ratio of the depth after erosion to the depth before erosion is called the erosion factor. Its allowable magnitude will depend on the type of the pier bases and must not exceed

$$P_e = \frac{h_e}{h_{en}} \leqslant 1.5 \tag{127}$$

To estimate the bridge opening using formula (125), it is necessary to measure the width of the river bed B_{en}, choose the type of bridge design according to the project, estimate the constraint of the flow by the bridge piers (λ, μ), select the erosion factor P_e (within the range from 1 to 1.5) and determine the ratio of the full river flow Q to the natural flow in the river bed Q_{en}. This ratio can be found approximately by morphometric means, i.e., by using the equation of uniform motion. The ratio of the flows can be replaced by the ratio of flow characteristics

$$\frac{Q}{Q_{en}} = \frac{\Sigma S}{S_{en}} \tag{128}$$

where $\Sigma S = \Sigma AC\sqrt{h}$ = total flow characteristic of the river bed and flood plain at the highest design water level
S_{en} = natural river bed flow characteristic
C = coefficient of Chezy formula.

The magnitude of the coefficient C for calculating the openings of large bridges can be determined more accurately by the formula

$$C = mH^{1/6} \tag{129}$$

where m is a coefficient allowing for bed roughness, the values of which for various beds are as follows:

Nature of River Bed	Value of m
Smooth earth beds; nonovergrown flood plains	30
Tortuous earth river beds; smooth beds of blind creeks; badly maintained earth ditches; flood plains overgrown by 10%	25
Highly tortuous earth river beds; tortuous or overgrown blind-creek beds; flood plains overgrown by 20%	20
Flood plains overgrown by 50%; highly overgrown blind-creek beds littered with stones	15
Fully overgrown blind-creek beds; boulder-strewn beds; mudflow streams; flood plains overgrown by 70%	10
Flood plains overgrown by 100%	5

38. Control of Roadbed Water Conditions

The roadbed serves as a base for the pavement and takes up all the pressure of traffic. Since soil resistance varies extensively with variation of its moisture content and the extent of compaction, in order to ensure a stable pavement it is necessary that the roadbed water conditions be maintained as constant as possible throughout the whole year.

One of the main means of ensuring the constancy of the roadbed water conditions is to prevent the penetration along the capillaries of surface water accumulated near the road, or ground water. The velocity of the capillary water flow depends on the type of soil and the degree of its compaction; the higher the degree of soil compaction, the slower the penetration of capillary water.

The duration of surface water ponding adjacent to the roadbed differs with various climatic conditions: in the northern regions of extensive saturation and on road stretches where drainage is inadequate water ponds almost always remain in side ditches; in southern arid regions water ponding occurs only for a short time since evaporation is rapid. Therefore, when establishing the elevation of the road pavement base along the route centre line, account should be taken of the probable period of time during which the water may penetrate into the roadbed. The ponding of surface water next to the embankment should be considered prolonged if it remains for more than 20 days.

The elevation of the roadbed above the ground-water table, or the level of prolonged ponding, is established for different climatic zones. Account is taken of the maximum capillary rise in compacted soils which is possible in these climatic conditions, of the duration of a high water table, and, for countries having a cold climate, of the duration of frost during which the winter movement of moisture and ice accumulation are probable (Tables 14 and 14a).

TABLE 14

Roadbed soils	Road climatic zones			
	II	III	IV	V
	Minimum elevation of pavement base above design ground-water table, m			
Medium and fine sands, light sandy loams	0.7	0.6	0.5	0.4
Silty sands, heavy sandy loams	1.2	0.8	0.8	0.7
Silty and heavy silty sandy loams; light, light silty and heavy silty loams *	1.9	1.7	1.4	1.3
Heavy loams, silty, sandy and rich clays	1.9	1.4	1.1	1.0

* The erection of high embankments of these soils is tolerated only in exceptional cases, if this gives a substantial reduction in costs.

TABLE 14a

Roadbed soils	Road climatic zones			
	II	III	IV	V
	Minimum elevation of pavement base above surface of ground on stretches with inadequate surface drainage			
Medium and fine sands, light sandy loams	0.5	0.4	0.3	0.2
Silty sands, heavy sandy loams	0.6	0.5	0.4	0.3
Silty and heavy silty sandy loams; light, light silty and heavy silty loams	0.8	0.6	0.5	0.4
Heavy loams, silty, sandy and rich clays	0.7	0.6	0.4	0.4

The amount by which the pavement base should be elevated above general ground level is given in Table 14a.

In cuttings, where drainage conditions are more difficult and drying out by wind is less effective, elevation of the pavement base above the ground-water table in silty and sandy soils should be 25 to 30% higher than that shown in Table 14.

Sometimes it is not possible to elevate the pavement base for individual stretches of the road to the height required in Table 14, e.g.,

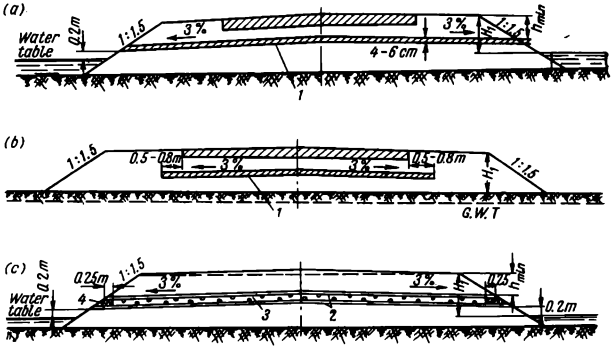


Fig. 70. Designs of isolating membranes:

a—impervious membrane with shoulders less than 2.5 m wide; *b*—ditto, when shoulders are wider than 2.5 m; *c*—capillary blanket course; *1*—soil treated with organic binders, or two layers of tar paper; *2*—anti-silting course; *3*—layer of gravel, rubble or coarse sand 20 to 30 cm thick; *4*—coarse gravel, rubble

when the road elevation is determined by that of an intersecting railway or a highway of a superior class. In such cases, in order to keep the water conditions in the upper part of the roadbed constant, impervious or intercepting membranes are laid within the embankment. These membranes, or courses, are located at a minimum height of 20 cm above the ground-water table.

By forming a barrier to water movement from the lower layers of the roadbed, these membranes induce favourable water conditions in the upper part, whilst the soil below this membrane may remain highly saturated. Percolation courses are made up to 30 cm thick depending on the size of the particles of the material employed (gravel, coarse sand). To prevent percolation course silting with finer soil particles, this is isolated from above and below by layers of soil composed of medium-size particles (sandy loam, fine gravel).

Impervious membranes (of bitumen, or bitumen-stabilized soil) inserted beneath permeable pavements are laid across the whole width of the roadbed (Fig. 70*a*). With very wide shoulders and impervious pavements the membranes are laid as closed-up strips (Fig. 70*b*). The performance of these is less reliable since any fissure occurring in the pavement permits water to accumulate in the closed volume of soil between the membrane and pavements.

A capillary blanket course (Fig. 70*c*) is used if near the road construction site coarse sand or fine gravel is available whose capillary rise is small. The thickness of the blanket course is from 15 to 20 cm, depending on the coarseness of material. To prevent the blanket material from inter-mixing with the roadbed soil, above and below the blanket a thin anti-silting layer 3 to 5 cm thick of moss, turf or straw is laid.

The further development of the chemical industry will lead to the use of thin polythene films as impervious membranes. In Great Britain, for instance, they are used in concrete pavement construction.

Impervious membranes and blanket courses are laid at a depth from the pavement surface not less than the following values:

Road climatic zone	II	III	IV	V
Depth to top of course, m	0.90	0.80	0.75	0.65

39. Drainage of Roadbed by Means of Land Drains

One of the ways of draining a roadbed which is becoming saturated with ground water is the laying of *land drains*, i.e., pipes laid in the soil, or ditches dug into the water-bearing ground which are filled with coarse filtering rubble, or simply open drain ditches.

The subdrain (Fig. 71) consists of a land drain (of tiles, ceramic, concrete, or wooden) laid in the ground, with small openings left for the admission of water. To prevent the drain from becoming blocked with soil it is surrounded by a porous filling whose coarseness decreases towards the walls of the trench. The porous filling intercepts water flowing from the ground, which is then discharged by the drain. In some cases, instead of a drain, stone rubble is laid, through the interstices of which the water flows.

Land drains can be used for lowering the ground-water table when the coefficient of permeability exceeds 1 m per day, and for intercepting the ground water flowing towards the road from outside. The effect of land drains is that the ditch or pipe laid in the ground below the ground-water table collects the water filtering from the adjoining ground. The result of this is that a dry zone is formed next to the land drain. The ground-water table next to the land drain becomes depressed; the intersection of it by a vertical plane at

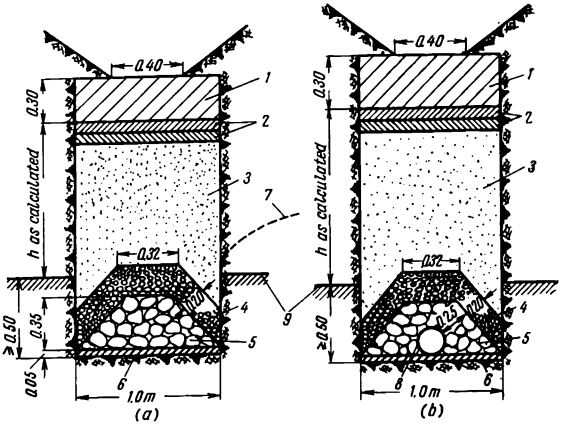


Fig. 71. Design of subdrains:

a—rubble filled; *b*—with drain; *1*—compacted clay; *2*—two layers of inverted turf or 3 cm of bituminous soil; *3*—coarse or medium sand; *4*—rubble or gravel, 5-10 mm; *5*—rubble or gravel, 40-70 mm; *6*—compacted rubble; *7*—water table curve; *8*—tile or asbestos-cement pipe, d=15-20 cm; *9*—confining bed

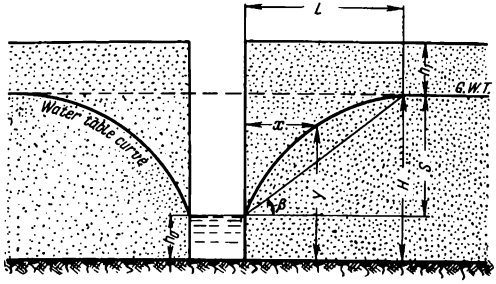


Fig. 72. Depression of water table at an open ditch

right angles to the ditch centre line forms a sloping water table curve indicating the profile of the *cone of depression* (Fig. 72).

The contour of the cone of depression can be obtained by the formula

$$y^2 = h_0^2 + \frac{2Q}{kL} x \tag{130}$$

where Q = inflow of ground water per unit length of the land drain, m^3/day

k = coefficient of soil permeability, m/day, the values for various soils being as follows:

Soil	Coefficient of permeability, m/day
Sand	1.5×10^{-2}
Sandy loam	0.6×10^{-2} to 1.5×10^{-2}
Loam	0.16×10^{-2} to 0.5×10^{-2}
Clay	0.5×10^{-4} to 1.6×10^{-2}
Young low-lying peat	0.16×10^{-2} to 0.75×10^{-2}
Middle-age low-lying peat	2.5×10^{-4} to 1.6×10^{-3}
Old low-lying peat	1.6×10^{-5} to 2.5×10^{-2}

The meanings of the other quantities in Eq. (130) are indicated in Fig. 72.

With a constant inflow of ground water, or of surface water penetration equal to that discharged by the conduit, the highest point of the cone of depression may be found according to the magnitude of the *angle of depression* β, which defines the permeability of the soil and its water yield. The slope of the water surface curve for various soils is characterized in the following table:

	tan β		tan β
Coarse sand	0.003-0.006	Loam	0.05-0.10
Sand	0.006-0.020	Clay	0.10-0.15
Sandy loam	0.02-0.05	Heavy clay	0.15-0.20

During intensive rainfall the water table curve rises, whereas during a drought it lowers. For determining the necessary depth of trench one should remember that the conduits draw off only the free ground water. The capillary water rise will follow a contour parallel to the lowering of the ground-water table.

Since the bearing strength of cohesive soil falls sharply when the capillary moisture content reaches its maximum, the effect of the drainage arrangement should be judged in practice in accordance with the depression of the capillary rise.

In the simplest case of the drains being laid below the ground-water table on the confining bed ("perfect" drain) the distance be-

tween the drains can be obtained from the formula

$$L = 2(H - S)\sqrt{\frac{k}{a}} \tag{131}$$

where H = original height of the ground-water table above the confining bed, m
S = required depression of the ground-water table, m
k = coefficient of soil permeability, m/day
a = maximum intensity of rainfall, m/day, obtained by dividing the amount of rainfall during the most rainy month—according to meteorological station data—by the number of rainy days.

In the case of "imperfect drains", i.e., not attaining the confining layer, the required distance is determined by the following formula derived by Soviet scientist Kostyakov

$$(H - S) = \frac{a}{2\pi K} L \log_e \left(\frac{L}{d}\right) \tag{132}$$

where d is the diameter of the drain. The remaining designations correspond to those in formula (131). The value of L is determined by selection, in accordance with this formula.

The arrangement of land drains near the roadbed for lowering the ground-water table is relatively inefficient. From the point of view of drainage the best practice will be to locate the land drain along the centre line of the carriageway. However, its maintenance and renewal when silted inevitably results in damage to the pavement. The digging up of land drains beneath the shoulders also weakens the pavement. Therefore, it is usual to construct two parallel drains under the shoulders or the inner slopes of the ditches. If the drain is situated directly beneath the ditch it has to be properly protected against silting by clayey particles carried by water penetrating from the side ditches. For ease of cleansing land drains situated beneath the inverts of the side ditches which serve for abstracting the water from the sand-gravelly anti-frost heave layer, inspection pits are provided approximately every 50 m.

It may be assumed that with few exceptions (cuttings in water-saturated soil) the use of land drains for lowering the ground-water level is the least efficient means for improving the ground flow network.

The simplest type of drain arrangement are open channels and wooden inverts whose bottoms are situated below the natural ground-water table. Wooden inverts are used to reduce the area taken up by the drain channels, which would otherwise require easy slopes to ensure their stability.

The longitudinal discharge from the drainage system towards low-lying country has to be assured. The minimum self-cleansing gradient at which no settlement of small soil particles and drain silting will occur, is as follows:

For pipes of diameter up to 200 mm	0.002 in clayey and 0.003 in sandy soils
For pipes of diameter 200 to 300 mm	0.0015
For pipes of 300 mm diameter	0.0005
For rubble filled drains	0.005

Intercepting drains are of greater importance to highways, being designed for the collection of the ground water percolating along the permeable layers which are traversed by cuttings, or which peter out on the surface above roadbeds on a hillside. Drain constructions of a similar nature to those used for cutting through a permeable layer are employed and sunk into the confining stratum. The permeable filling should cut across the whole depth of the permeable layer. The inner wall of the drain trench is made as an impermeable barrier of highly compacted clayey soil or puddle clay (Fig. 73). The intercepting drains are designed to collect all the water seeping along the permeable bed.

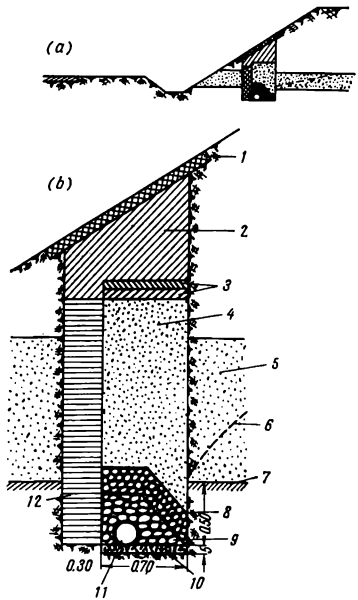


Fig. 73. Design of an intercepting drain:

a—general arrangement of drain on hillside; *b*—detail of drain; *1*—turf; *2*—compacted clayey soil; *3*—two layers of turf or bituminous soil; *4*—coarse or medium sand; *5*—permeable layer; *6*—water table curve; *7*—confining bed; *8*—rubble or gravel, 5-10 mm; *9*—rubble or gravel, 40-70 mm; *10*—rubble rammed into the soil; *11*—asbestos-cement or tile drain, *d*=0.15-0.20 m; *12*—screen of kneaded clay

CHAPTER 7

DESIGN OF ROADBED

40. Stability Requirements for Roadbed

With the high speed of modern motor vehicles it is essential that there should be no marked deterioration in the riding quality of the road between maintenance operations. This is possible only with a stable roadbed which is not subject to differential settlement or to the processes of frost heave formation.

When the roadbed is constructed the stability of the upper layers of the earth crust is often impaired. Excavations through steeply-dipping soil strata during the formation of cuttings may cause slipping of the faces. Embankments formed on the side of a hill, i.e., in sidelong ground, may tend to creep (Fig. 74*a*). Peat and saturated clay beds can be extruded from under embankments (Fig. 74*c*) or slowly consolidated by the additional weight of the embankment, through the considerable reduction in water content, and hence in void ratio (Fig. 74*b*).

The consolidation of soil beneath high embankments often causes damage to culverts situated under them, owing to the greater inherent settlement at the middle of the embankment than at the edge.

Apart from the lateral displacement of embankments as a whole, settlement of the embankment itself is possible, caused, for instance, by consolidation of the fill under the influence of natural factors, the dead weight or impact loads and vibrations due to traffic (Fig. 74*d*). The deformation due to soil consolidation is possible not only in embankments, but also in cuttings and on stretches following the natural profile, if the natural density of the soil as found is insufficient. The loss of stability by the roadbed can also manifest itself by the warping of the road profile owing to slumping (Fig. 74*e*).

The roadbed stability is inseparably linked with the water conditions. An appreciable increase in the moisture content of the soil beneath the road will lower its cohesion and may cause deformations in the part of the roadbed subject to saturation.

The saturation of the roadbed slopes by rain or flood water may be the cause of washouts and slipping of the side slopes. Embankments filled with wet soil which includes unbroken lumps may become oversaturated and slide down.

The experience of road and railway building over a very considerable number of years has enabled stable roadbed designs to be

worked out for stable geological conditions—the so-called *standard road cross-sections*, which are illustrated in the engineering specifications of various countries. However, in difficult ground conditions, and when high embankments have to be built, one cannot

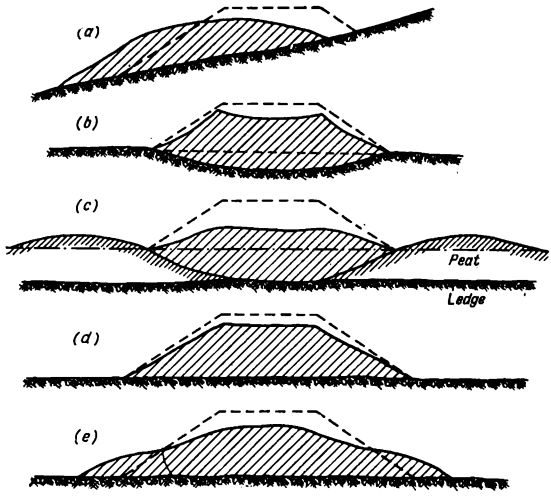


Fig. 74. Kinds of roadbed deformation:
a—embankment slip down hillside; *b*—settlement due to subsoil compaction; *c*—extrusion of peaty or silty subsoil; *d*—settlement due to soil consolidation inside embankment; *e*—deformation due to spreading of saturated soil

rely on typical solutions and has to resort to an individual design to ensure stability of the roadbed.

Methods for determining the stability of the roadbed fall within the province of soil mechanics. Nevertheless, one has to keep in mind that the service conditions of roadbeds are more complicated than those of civil and industrial structures.

The soil within the roadbed is subject to fluctuations in moisture content and to temperature variation in time and magnitude. As a result the ground resistance to loading is not constant throughout the year, whilst annual weather conditions also vary, not being the same from year to year. Thus, the strength of the roadbed is varia-

ble, and for this reason one has to design for periods when ground conditions are the most unfavourable.

The deformation of the roadbed depends to a great degree on the uniformity of granulometric composition or grading and the extent of soil compaction, on the distribution of moisture in the roadbed, on the soil temperature and on a series of additional factors, the assessment of which may be difficult. Stability calculations are inevitably related to the basic assumptions of soil deformation and to assumptions of soil uniformity within the limits of the separate layers.

In some cases stability calculations based on average conditions within the roadbed are not necessarily relevant to all parts of these structures. In particular, one cannot fully guarantee the stability of side slopes throughout the economic life of the road. The soil on the surface of the side slopes is subject to the action of climatic factors which cause weathering and reduction of strength of the covering layers. The action of the weathering factors manifests itself differently according to the orientation of the slopes in relation to cardinal points. In countries within the Northern hemisphere the natural slopes facing South are always steeper than those facing North, because in springtime they are the first to be cleared of snow and in summer they dry out more rapidly after a rain. For this reason, when computing the stability of the roadbed one should not neglect the experience gained from the operation of existing roads and railways, laid in the vicinity of the new route.

The extent of the resistance to sliding, settlement and other deformations is defined by the *stability number*, which is the ratio of forces, or their moments, retaining the embankment to those forcing it to shift.

The roadbed stability numbers for motor roads are not standardized. For calculation purposes the numbers similar to the ones recommended for hydraulic structures can be accepted, and are chosen according to the construction purpose. The stability numbers are determined for the least favourable values and combinations of forces and loads acting on the structure and its foundation under normal building and operation conditions. The minimum stability numbers should be as follows:

	Stability number
Permanent structures meeting high requirements	1.3-1.4
Ditto, average requirements	1.25-1.3
Ditto, below average requirements	1.2
Temporary structures	Not standardized

The calculation of soil stability for earth structures, and for structural foundations, requires a reliable knowledge of the character-

istics of soil strength, obtainable only as a result of laboratory tests or those carried out directly in the field.

U.S.S.R. building standards and regulations recommend that the soil characteristics used for calculating the foundation deformation and stability (stress-strain modulus, Poisson's ratio, angle of internal friction, cohesion) should be determined in accordance with soil tests upon undisturbed samples so far as practicable.

The resistance of soils to loading differs sharply with the degree of soil compaction and their moisture content, and also as a result of the action of natural factors during the service life of the structure. This is why the design characteristics of soil strength, as a rule, have to correspond to the least favourable operational condition of the foundation.

An estimation of permissible soil stresses most suitable for a structure has to be made for each particular case after a thorough analysis of the purpose of the structure and of the local geophysical conditions. At the same time one has to bear in mind that as a result of erecting the structure or of earthworks the condition of the soil foundation may radically change, as well as the moisture content and temperature conditions. An example of such a case is the weathering of clay and shale rock on the side slopes of cuttings, also the alteration in the character of a swamp as a result of consolidation due to the embankment deadweight forming a barrier obstructing the natural flow of ground water.

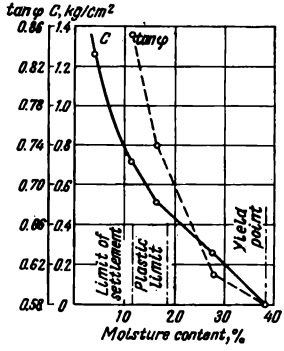


Fig. 75. Example of graph showing design soil characteristics

The characteristics of soil strength, e.g., cohesion c, angle of internal friction φ, stress-strain modulus K, unit weight δ, etc., substantially depend on the soil moisture content and the degree of soil compaction. When estimating the stability of a structure graphs of design strength characteristics have to be drawn in order to assess precisely the conditions of the soil in accordance with the laboratory tests. On these graphs curves are plotted for saturated soils relating moisture content and variation in consolidating pressure. An example of such a graph is given in Fig. 75.

When choosing the method of soil testing special attention should be given to the test conditions being as close as possible to the opera-

tional condition of the soil beneath or in the structure. The soil should be tested in a state corresponding to its performance under the most unfavourable condition for stability and the character of soil deformation should correspond to operation conditions. Thus, for instance, the study of the pervious nature of soils for the design of sub-drainage should be carried out on undisturbed samples. The rate of water filtration through a roadside dam, made of the same soil, should be assessed by using samples compacted to an optimum density and after remoulding.

When determining the coefficient of permeability of peat beneath an embankment one should take into account the direction in which the water drains from under the embankment, since the peat coefficient of permeability for water percolation varies according to whether drainage is in a vertical or horizontal direction when its texture is undisturbed.

Subsoils in their natural condition are usually heterogeneous and occur with random stratification. However, the physical and mechanical properties, even of strata having uniform texture, differ in various conditions since their moisture content alters.

The rated values of soil characteristics should be determined according to tests of a sufficiently large number of samples, in order to obtain satisfactory average values. The number of tests should be the greater, the more important the structure.

The selection of separate soil strata differing in their properties can best be made graphically, plotting the points of the experimental soil characteristics in relation to the sampling depth. The grouping of these points enables one to bring out characteristic stratification.

The soil characteristics used for preliminary calculations can be those used in formulas for stability calculation. Tables 15 and 16 give characteristics of clayey and sandy soils according to Prof.

TABLE 15

Soil texture	Clay			Loam			Sandy loam		
	γ g/cm³	φ degrees	c kg/cm²	γ g/cm³	φ degrees	c kg/cm²	γ g/cm³	φ degrees	c kg/cm²
Hard	2.15	22	1.00	2.15	25	0.60	2.05	28	0.20
Medium hard	2.10	20	0.60	2.10	23	0.40	2.00	26	0.15
Hard plastic	2.05	18	0.40	2.00	21	0.25	1.95	24	0.10
Soft plastic	1.95	14	0.20	1.90	17	0.15	1.90	20	0.05
Flow plastic	1.90	8	0.10	1.85	13	0.10	1.85	18	0.02
Fluid	1.80	6	0.05	1.80	10	0.05	1.80	14	0.00

TABLE 16

Sand designation	Unit weight of sandy soil δ, g/cm³		Angle of internal friction of the sandy soil φ, deg.	
	Medium density	Dense	Medium density	Dense
Fine powdery	1.92	2.00	26	30
Fine	1.92	2.00	27	30
Medium	1.94	2.00	28	32
Graded	1.96	2.05	29	33
Coarse	1.98	2.05	29	33
With gravel and pebble	2.00	2.10	30	35

N. N. Maslov. The solutions obtained should be checked at the final design stage by using the soil characteristics obtained by laboratory tests of soil samples having undisturbed texture and a moisture content corresponding to the less favourable condition of operation.

According to their use for the construction of roads soils are classified as follows.

41. Disposition of Soils in a Roadbed

Rocky and *gritty soils* are debris of rock either of natural origin or obtained as a result of artificial processes, e.g., crusher-run. These soils constitute good material for filling embankments, since they are resistant to the action of flowing water and do not absorb moisture. Water penetration into the interstices between the debris has no substantial influence on the strength and stability of the resultant roadbed. Some weak and easily-weathered rocks are exceptional in this respect, e.g., marl, clayey shale and chalk, which tend to disintegrate or swell when saturated with water. These materials can be used only in the lower layers of embankments in dry localities, which are not subject to prolonged periods of saturation. In the upper layers and on the sides of embankments these soils should be covered by a layer of impervious soil at least 1 metre thick to prevent them from absorbing rain and thaw water.

Gravel and *sandy soils* are pervious to water. Water saturation has little effect on the stability of these soils in the roadbed. Sandy soils are the best material for embankments constructed under unfavourable hydrogeological conditions, e.g., in swampy regions and on fluvial plains. Owing to a reduced capillary rise and excellent water permeability these soils do not normally become satu-

rated in a pavement subgrade and are quick to dry out in flooded embankments once the flood waters subside.

Because of the limited resistance to erosion the slopes of sandy embankments must be specially reinforced where water is discharged down their sides. Moreover, the slopes of these embankments and cuttings have to be stabilized against erosion by rainfall washout and wind.

Sandy loam contains a small proportion of clay particles, sufficient to give them cohesion in the dry state. When moistened the sandy

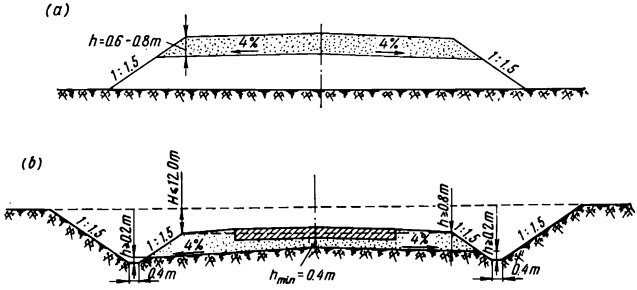


Fig. 76. Replacement of unstable soils:
a—in embankments; b—in cuttings

loam retains a sufficient resistance to loading to provide for the stability of the roadbed. Embankments of sandy loam can be constructed in dry, as well as in very wet localities.

Fine sandy loam, containing over 50% of particles finer than 0.25 mm, is less stable in the saturated state.

Powdery soil, and also silty or sandy loam contain a larger proportion of fractions between 2.0 and 0.05 mm. These soils are easily eroded on cutting and embankment slopes, where under adverse conditions they will flow.

When constructing a roadbed for a road with a carriageway to be used for high-speed traffic, when hydrological conditions are unfavourable and the soil is either powdery or a silty loam with a high moisture content, the top 1.2 m (in Zone II) or 1.0 m layer (in Zone III) of an embankment with cement concrete pavements and, respectively, 1.0-0.8 m with asphalt concrete pavements, counting from the pavement surface, should be filled in with sandy and sandy loam soils. In cuttings the top layers of the soil are replaced to the same depth with stable soils (Fig. 76).

Loam soil is an excellent fill material for the roadbed. Its resistance to erosion is high and it is stable on the side slopes. On flooded loam-filled embankments the seepage of the contained water through the embankment following a fall in water level may cause a hydrodynamical pressure leading to the collapse of the slopes.

Argillaceous soils have a high cohesion and a very low permeability, are thus slow to absorb water and just as slow to dry out. These soils may be used to fill embankments in dry places and in localities of limited saturation and when the moisture content of the filling material does not exceed 1.10 of the optimal value with standard compaction. In a saturated state the argillaceous soils become very plastic or fluid, are sticky, and cannot be compacted. In countries with a hot arid climate and having periods of prolonged rainfall the argillaceous soils in a roadbed are prone to shrinkage and swelling which may lead to the destruction of the pavements.

Organic soils—silt and peat—have a marked tendency to considerable volume changes (swelling and shrinking) with the variation of moisture content. Wet silty soils lose their cohesion and become unstable. Peat is highly compressible and cannot be used for construction of embankments.

The content of soluble salts and organic materials in soils (sa ine soils, peat and partially chernozem) may substantially alter their physical and mechanical properties. The special requirements of road design in such soils are examined in the section dealing with the design of roads in difficult geophysical conditions.

When the ground is excavated by earth-moving machines its natural structure is destroyed and the soil is broken up into separate lumps. Unless the lumps are broken up and the soil in the embankment is adequately compacted, water will percolate through the voids between the lumps and rapidly lead to saturation and instability.

The inclined layers of cohesive soils occurring within the embankment—especially if these were compacted by the passage of road machines—may behave, when wetted, like a lubricated surface along which part of the embankment may slide. For this reason, when using soils of varying composition and texture, it is necessary to comply with definite rules of soil distribution within the embankment in order to ensure stability of the roadbed (Fig. 77). For this purpose the following requirements are to be complied with:

1. Heterogeneous soils should be placed in horizontal layers within the embankment.

2. Impervious soils are deposited in layers with two inclined surfaces having a transverse slope of 4%, which provides for the discharge of percolating water. There should be provision for water discharge from pervious soils.

3. The embankments should not be filled in such a manner as to form a closed core consisting of one kind of soil, covered above and on the sides with a different kind. An exception are the cases

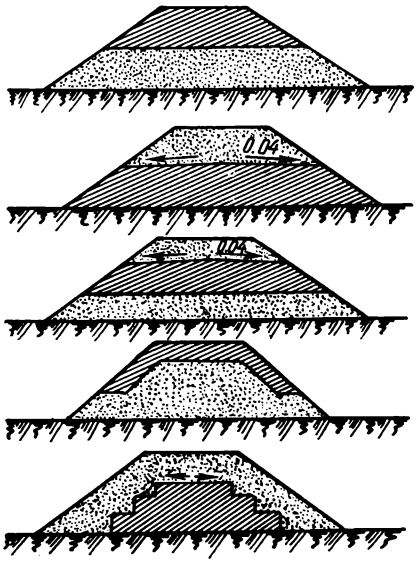


Fig. 77. Rules for locating permeable (dotted) and impermeable (cross-hatched) soils in roadbed

mentioned above, concerning the use of soils which become unstable under the influence of water, and also cases of roadbed widening during the reconstruction of roads. In the latter case special measures should be taken to ensure the stability of the added strip.

42. Stability of the Road on Hillsides

An embankment constructed on a hillside may slide down if the component of its weight acting parallel to the slope exceeds the maximum frictional force which retains the embankment (Fig. 78). In cuttings the hillside may slide as a result of excavation into

steeply-dipping rock strata, and also because of excessive gradient in the case of homogeneous rock.

According to Fig. 78, the retaining force is

$$R = fQ \cos \alpha$$

where Q = weight of the embankment
f = coefficient of friction of the fill laid on the hillside
α = angle of slope.

The force tending to displace the embankment is

$$F = Q \sin \alpha$$

The coefficient of embankment resistance to shear is

$$K = \frac{R}{F} = \frac{Qf \cos \alpha}{Q \sin \alpha} = \frac{f}{i} \qquad (133)$$

where i is the transverse gradient of the hillside expressed as a decimal.

The stability of the embankment can decrease during the rainy season of the year, when the water running down the dense soil

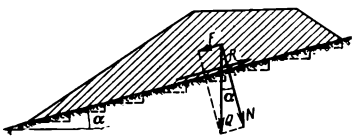


Fig. 78. Forces acting on embankment on hillside

surface and percolating under and into the embankment moistens the lower layers of the filling in the zone of contact with the bed soil. The saturated soil loses its cohesion and forms a layer with a reduced resistance to shear, along which the embankment may slide.

The measures taken to enhance the stability of the embankment on slopes are designed to increase the value of the coefficient of friction f.

When the transverse gradient of the land is from 1/10 to 1/5 it is necessary to remove the turf from under the embankment. With the transverse gradient exceeding 1/5, benches must be cut into the hillside with the aid of excavators, bulldozers or autograders. The benches are made 0.4 to 1.75 m high, depending on the steepness of the slope, and are given a transverse gradient ranging from 2 to 3% falling in the direction of the slope.

The width of the benches depends on the length of the grader blade. When excavators or bulldozers are used it is about 3.5 m, when using autograders it is from 2.5 to 3.0 m, and when made manually it is 1 to 2m. The aim of benching is to substitute the higher resistance to shear of the filled up ground on the surface of the slope for the inadequate soil resistance to sliding along the same surface. It is assumed that the hillside subsoil strength is such that shear can occur only through the cross-section of the filled up ground.

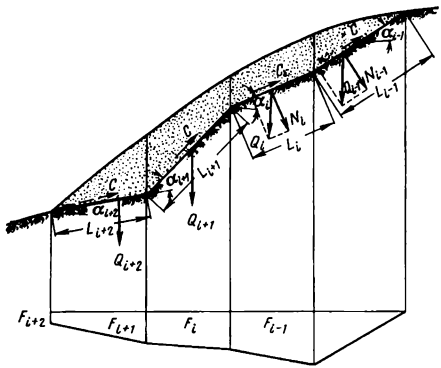


Fig. 79. Determining stability of slope on compact bedrock

With weak soils, in particular on slopes composed of sand or loosely cohesive gritty soil, benching does not provide a solution. In such cases, to provide for adequate stability of the embankment, it is necessary to construct retaining walls or counter-benches.

In the case when the embankment is laid on an inclined bedding, occurring on stable rock (Fig. 79), the embankment may slide under the combined action of its own weight and that of the underlying subsoil. A similar phenomenon may occur when a side slope of a cutting is incised into steeply-dipping soil strata.

The stability check is made by trial and error evaluation of the resistance to shear of successive sections of the sloping layer under the influence of the pressure from adjacent sections.

The sections used for calculation are selected according to the nature of the slip surface profile. For each individual section, which is considered as an isolated mass, the projections of all the acting forces on the sliding surface are summated.

The magnitude of the pressure exercised by the i-th section on the following section is

$$F_i = F_{i-1} \cos(\alpha_{i-1} + \alpha_i) - Q_i \cos \alpha_i \tan \varphi + Q_i \sin \alpha_i - cL_i \quad (134)$$

where F_{i-1} is the pressure transmitted from the section above to the examined section.

If for the section above the force F_{i-1} has a negative value, then the value of F_{i-1} is ignored in the calculation.

The stability number of each section is

$$K_i = \frac{Q_i \cos \alpha_i \tan \varphi + cL_i}{F_{i-1} \cos(\alpha_{i-1} + \alpha_i) + Q_i \sin \alpha_i} \quad (135)$$

Examining successively the stability conditions of a series of sections, it is possible to determine the places where ruptures are most likely to occur. These correspond to the sections with a minimum value of K, where bulging of the formation may be anticipated.

43. Degree of Consolidation and Settlement of Roadbed

Under the action of its own weight and of the alternate moistening and drying, the soil in the embankment which was loosened during the excavation will gradually consolidate. Several wet seasons may be necessary to complete its final settlement. Comparatively recently the embankments used to be filled allowing an additional margin for settlement, which took several years to complete. With modern methods of high-speed road building it is not expedient to wait for the completion of a prolonged period of natural soil consolidation in the embankment. To preserve the smoothness of the road surface, the soil in the roadbed is artificially compacted during the process of filling. However, the modern mechanical means are not yet sufficiently effective, and for attaining an adequate compaction, very many passes are necessary; or alternatively compaction must be carried out in thin layers. Therefore, for pavements which are very sensitive to subsoil failure, it is recommended that the roadbed be completed a year before the construction of the pavement, thus combining artificial compaction with natural consolidation.

The extent of soil consolidation in the embankment, which limits the degree of settlement or internal strain, depends on the stresses in the ground. The *extent of soil compaction* can be measured in terms of the unit weight of the solid phase (skeleton) of the soil. Thus, to prevent subsidence inside the embankment due to natural consolidation of the soil, its porosity should correspond to the equilibrium condition for the confining intergranular pressure (Fig. 80). The

stresses due to ground dead weight are proportional to the depth of the layer under consideration. The outside load gives rise to stresses which are dissipated effectively at a certain depth below the

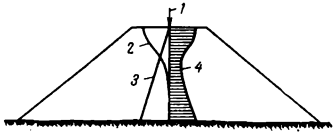


Fig. 80. Distribution of stresses in embankment:

1—load; *2*—stress induced by external load; *3*—stress induced by weight of soil; *4*—diagram of total stresses

surface. For low embankments the impact loads of vehicles are transmitted into the subsoil and may cause its subsidence if its compaction is inadequate (Fig. 81). This is why loose arable and

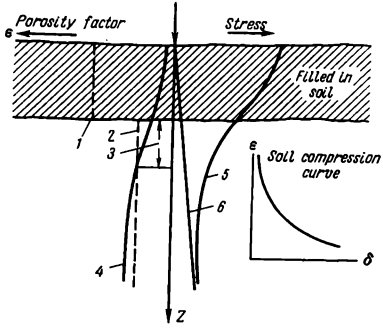


Fig. 81. Determination of depth to which bed soil under a high embankment must be compacted:

1—porosity factor of loose filled-in soil; *2*—porosity factor of soil in natural bedding; *3*—zone within whose limits the bed soil will be compressed by the action of the load; *4*—porosity factor corresponding to δ_z; *5*—total stress σ_z in soil; *6*—stress due to weight of soil

subsiding loessial soils must be thoroughly compacted with heavy large-diameter rollers, the action of which is transmitted to a considerable depth, before the construction of the roadbed. In the

lower part of flooded embankments capillary pressure may be present, as well as shrinkage stresses when the soil dries after the subsidence of high water.

The extent of soil compaction for each zone is determined in accordance with the nature and degree of the stresses occurring within its limits.

In the topmost layer of an embankment, to a depth of 1.2-1.5 m, static and dynamic stresses are induced by moving vehicles. Also the processes of moistening and drying out of soil, due to the influence of the annual variation in the flow network, are intensified. The compaction of soil in this zone should correspond: for cohesive grounds—to the pressure of internal forces causing the settlement; for sandy loam, soft loam and sand—to traffic stresses. Investigations have shown that for each natural region there is a certain optimum degree of soil compaction in the zone of moisture migration. In climatic regions where intensive moisture migration occurs, soils which have been excessively compacted during the construction period are prone to become less compact after several years of operation.

In the embankment middle layers within the depth range extending to 10 m from formation level the soil water conditions are comparatively constant, and the external load and ground dead weight stresses are small. Therefore, within the range of this zone a lesser degree of compaction is acceptable.

In the lower embankment layers on stretches prone to flooding and to periodic capillary moistening and subsequent drying, the requirements for the degree of soil compaction are similar to those for the embankment upper layers.

The lower embankment layers constantly located below the water table work in conditions of compression under the influence of the weight of the embankment layers above them and of the external load. In these layers a degree of compaction sets in that corresponds to a compression relation.

In the lower embankment layers not flooded with water the degree of compaction may be similar to that in the middle layers.

The required degree of soil compaction within the limits of each layer can be determined according to soil compression tests using a moisture content typical of the operational conditions, and which are based on actual loading but allowing for its repetitive cycle. For estimating the required compaction one must rely on experience gained from the investigation of old embankments. The requirements for compaction are standardized in accordance with the density of the undisturbed sample and are expressed in fractions of so-called *optimum compaction* determined in the laboratory at the *optimum moisture content.*

The soil optimum moisture content is the one at which soil compaction can be attained with a lesser effort than that required for other moisture contents. This moisture content approaches the mean value of soil moisture content in borrow pits during the period of excavation.

It is known from experience, that when a sample of soil is compacted in a standard compaction apparatus by means of a weight falling from a constant height, 3 to 5% of air still remains within the soil pores. This enables the degree of optimum compaction to be determined experimentally as well as by calculation.

Suppose that a unit volume of soil compacted in a standard apparatus contains a proportion m of solid phase, w_0 of water, and v of air, these values being expressed in fractions of unity.

The volume of the solid phase can be expressed by means of the unit weight of the skeleton (solid phase) δ and the specific weight of the soil γ

$$m = \frac{\delta}{\gamma} \tag{136}$$

The volume occupied by water is the product of the proportion of water expressed in fractions of unity and the unit weight of the soil skeleton

$$w_0 = w\delta$$

As stipulated

$$\frac{\delta}{\gamma} + \delta w + v = 1 \tag{137}$$

and, therefore, the weight of the soil per unit volume is

$$\delta = \frac{\gamma}{w\gamma + 1} - \frac{v\gamma}{w\gamma + 1} = \frac{\gamma(1 - v)}{1 + w\gamma} \tag{138}$$

Formula (138) can be expressed as

$$\delta = \frac{\gamma}{w\gamma + 1} - \alpha$$

where $\frac{\gamma}{w\gamma + 1}$ = unit weight of soil with pores fully saturated with water

α = weight of water in the pores partially filled with air.

The value of α differs according to the requirements which the examined soil layer is to comply with. These values are established experimentally.

The requirements for the degree of compaction δ_r of soil in the roadbed are also expressed in relation to the optimum compaction

$$\delta_r = k\delta_0 \tag{139}$$

where k is the optimum compaction factor.

For the construction of highways the roadbed should be compacted to not less than 0.95 of the optimum compaction factor, at least within the upper layers of embankments.

A high degree of compaction of the roadbed soil will not only limit the amount of subsequent settlement, but also further the stabilization of the roadbed water conditions. The velocity and the height of capillary rise in the roadbed decrease as the degree of soil compaction increases. The capillary moistening of compacted soils does not lower their resistance to external loadings. Such a reduction may be caused only by an increase in the void-ratio of the soil owing to its swelling under the prolonged action of water, and also winter frost action and the formation of ice lenses. For this reason the compaction of soil should always be complemented with measures providing for adequate drainage.

The denser the ground, the slower is the process of moisture transfer and, therefore, the shorter are the periods of excessive saturation (due to rain, snow thaw and floods).

44. Stability of the Roadbed on Weak Bedding Soils

Embankments constructed on weak bedding soils (loose soil, peat, plastic saline soil, loessial soil, water-saturated silty soil) will subside owing to settlement of the bedding under their dead weight. This settlement can be caused by the consolidation of the bedding or by the lateral displacement of material from under the embankment. Most of this settlement usually occurs during the early period following the construction of the embankment, but in some cases it can take place during the operation period, e.g., during prolonged standing of heavy loads on embankments erected on a swamp or during the ponding of water next to the embankment—on sensitive soils.

In practice, when designing structures and buildings the loading on the ground is usually distributed in such a way that the settlement does not attain a value greater than that allowed by the relevant standards.

The load of the roadbed on the bed soil depends on the height of the embankment, which is determined by the location of the grade line. It is therefore necessary to assess the resistance of the roadbed to sudden subsidence, and, if such sudsidence and settlement are in-

evitable, to calculate their magnitude in order to compensate by means of additional filling during the construction stage.

The relation between the pressure on soil and its settlement is shown in general form by the curve in Fig. 82. Individual sections of this curve characterize various phases of the deformation process. Within the limits of a section, where the relation between the loading and the strain is approximately rectilinear, consolidation of

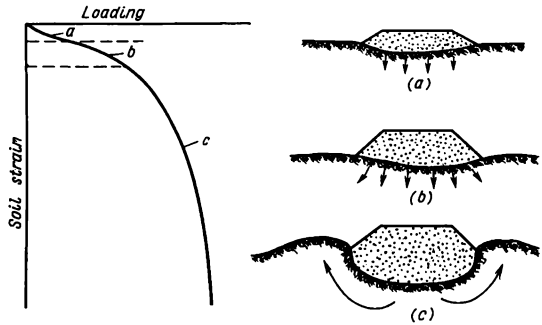


Fig. 82. Relation between soil strain and loading on soil:
a—consolidation; b—consolidation and side shift; c—extensive subsidence caused by bed soil bulging out sideways. The arrows indicate the direction of soil shifting due to strain

the bedding course is the main phenomenon. As the load increases, in certain points of the bedding course the lateral strain begins to exceed the resistance to shear. In these places *plastic deformation* (*shear strain*) occurs. As the loading stress increases such places become more numerous, the embankment settlement increases and the *extrusion of material from under the embankment* takes place, accompanied by the formation of mounds at the sides of the embankment and further subsidence.

Thus, the nature of the settlement of the bedding course depends on the pressure exercised by the embankment and on its resistance to external loads. The extrusion of homogeneous soil, from beneath the embankment, when occurring at an appreciable depth, is accompanied by the formation of curvilinear sliding surfaces. During an embankment subsidence the bed soil may bulge out on one or both sides.

When constructing embankments on soft bed soils, whose depth is small compared to the width of the embankment, the plastic

deformations extend over the whole layer of soft soil under the embankment, which is forced out laterally. The pressure which causes the lateral bulging of the homogeneous soil can be determined by soil mechanics formulas. The stability of the soil is deduced from the maximum equilibrium condition.

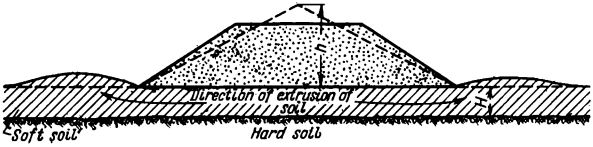


Fig. 83. Extrusion of thin layer of soft soil from under embankment

The limiting pressure of the embankment before the ground is forced from under it, is given by the formula

$$p = (\delta\Delta + c\cot\varphi)\frac{1+\sin\varphi}{1-\sin\varphi}e^{\pi\tan\varphi} - c\cot\varphi \qquad (140)$$

where p = limiting pressure on the soil, kg/cm², after which lateral displacement from under the embankment occurs
δ = unit weight of the soil
c = soil cohesion
φ = angle of internal friction
Δ = depth of settlement of the embankment.

If the pressure of the embankment on the ground p_1 exceeds the limiting value, then with the subsidence initially equal to zero lateral extrusion commences, ceasing when the subsidence attains a value Δ at which the pressure $p_1 = p$.

The method of calculation according to the maximum equilibrium condition can be used for cases when the depth of the layer of the deformed soil under the embankment is not less than 1.5 of the embankment width at its base. At a lesser thickness of the deformed layer, a method of assessing the resistance to extrusion of thin layers of weak soil should be used in order to estimate the maximum pressure (Fig. 83). This method was developed by the Estonian scientist L. K. Yurgenson and is based on the analysis of plastic flow of a layer of material compressed between two rigid surfaces. In this case it is assumed that:

1. The strength of the soil depends exclusively on its cohesion, and its angle of internal friction is nil. Such an assumption is admissible for saturated clay or organic silt (sapropel) with a moisture content exceeding the fluidity limit, but gives an appreciable safety margin for peat, whose angle of internal friction reaches 30°.

2. With the extrusion of soil from under the embankment shear may occur only within the layer of soft ground.

3. The forcing of soil from under the embankment is not accompanied by a change in its volume. This supposition corresponds to the case of the displacement of highly decayed peat, sapropel and saturated clay, all of which have a very small coefficient of permeability.

4. The bases of the embankment before and after the subsidence are parallel to the firm bottom. This applies only to embankments erected on a floor of logs (mat), which is admissible for roads of lower classes.

To allow for actual conditions it is assumed in calculations that the resistance to extrusion of weak bed soils under embankments having a parabolic base profile is only half of that under embankments constructed on a mat.

According to L. K. Yurgenson the critical loading which causes soil to be forced out from under the embankment with a horizontal base is

$$p = 2\frac{cb}{H} \tag{141}$$

where c = soil cohesion
b = embankment semi-width at its base
H = depth of the soft ground layer.

To increase the resistance of embankments to extrusion of bed soil at their base, the following measures can be taken:

(1) reduce the dead weight of the embankment by using lightweight materials, i.e., slag or peat; limit he height of the embankment by the insertion of impervious membranes;

(2) construct adjacent to the embankment, and parallel thereto, benchings whose weight will tend to counteract the bulging of the bedding course;

(3) transmit the weight of the embankment onto the bedding course through a pile foundation;

(4) construct the embankment on a mat which prevents the extrusion of bed soil and distributes the embankment pressure over a larger area.

The expediency of using these methods should be justified economically by comparing them with the normal solution, i.e., of constructing the embankment on a hard bottom.

The calculation of embankment settlement on compressible bed soil enables one to assess the additional quantity of earthworks necessary to compensate for shrinkage. The estimation of settlement of high embankments is also important for determining the construction height of culverts laid beneath. The middle sections of the latter, situated under the central part of the embankment, are most

affected by the subsidence, whilst those at the edges, over which the fill is insignificant, remain in place.

The calculation of embankment settlement due to soil shrinkage involves the determination of the stress distribution pattern in the bed soil according to depth and the summing up of deformations of individual layers due to vertical contraction.

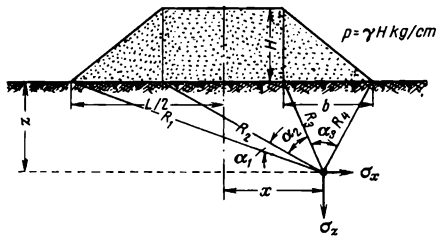


Fig. 84. Diagram for determining stresses in soil induced by weight of embankment

The distribution of stresses in the bed soil is determined according to formulas of the theory of elasticity for the case of a load applied to an infinitely long strip having a trapezoidal cross-section (a plane problem).

The value of the normal vertical stress in the bedding course σ_z due to the weight of the road embankment is determined by means of the formula

$$\sigma_z = \frac{p}{\pi b}\left[b(\alpha_1+\alpha_2+\alpha_3)+\left(\frac{z}{2}-b\right)(\alpha_1+\alpha_3)+z(\alpha_1-\alpha_3)\right] \qquad (142)$$

The notation used in this formula is pictured in Fig. 84. The angles for this calculation must be expressed in radians.

When calculating the settlement of weak subsoils, it is assumed that the consolidation of the soil may be neglected at a depth at which the stress due to the embankment weight becomes less than 0.1 of the soil dead weight pressure

$$\sigma_z < 0.1\delta z$$

For denser soils it is sufficient to assess the consolidation of the layer limited by the level at which the stress caused by the weight of the embankment becomes equal to pressure of the soil.

When calculating the soil weight pressure for layers below the ground-water table, allowance is made for the suspending effect of

the water. In this case the unit weight of the soil is assumed equal to 1 g/cm³.

The extent of settlement is determined by integrating the individual settlements of the various ground layers, within the limits of which the stress condition and the strain characteristics (modulus of strain for the soil, coefficient of compression) can be considered constant. The actual curve of pressure distribution is replaced in

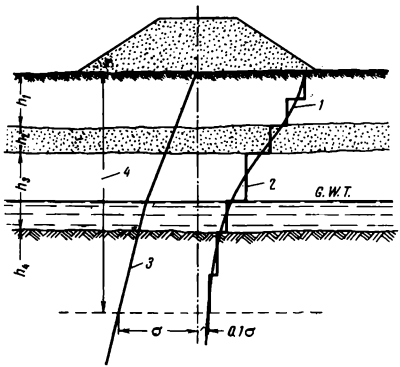


Fig. 85. Determining settlement due to consolidation of soil under embankment:

1—stresses induced by embankment pressure; 2—ditto, represented by a stepped diagram; 3—stresses caused by weight of soil; 4—depth of layer of soil being compressed

this instance by a stepped line (Fig. 85). The thickness of the individual layers should not exceed 0.4 of the embankment width at its base.

If the depth of the affected layers of ground is less than the width of the embankment at its base, which may occur at swamp crossings, the falling off of stress with depth is ignored and the settlement is determined as for a layer contracted by a pressure uniformly distributed throughout the full depth.

The consolidation of comparatively dense soils is characterized by the modulus of strain, the value of which is determined experimentally. In this case the contraction of an individual layer of thickness h is

$$\Delta = \frac{h\sigma_z}{K_s} \tag{143}$$

and the total settlement of the embankment is

$$\Delta = \sum_{i=1}^{i=n} \frac{h_i \sigma_{z_i}}{K_s} \tag{144}$$

where n is the number of individual layers.

If the relative contraction of the lower layer exceeds 0.1% (1 mm for a 1-m depth of soil), the calculation is continued and the consolidation of the underlying soil layers is assessed.

When calculating the settlement of an embankment on highly sensitive ground, e.g., on peat bogs or loose soils filled in during landscaping operations, one has to take account of their nonlinear strain and use the compressibility curve for the determination of settlements.

With the variation of the soil porosity factor from ε_1 to ε_2 the consolidation of the soil layer of a depth H becomes

$$\Delta = \frac{\varepsilon_1 - \varepsilon_2}{1 + \varepsilon_1} H \tag{145}$$

where ε_1 = soil porosity factor before the construction of the embankment at the natural pressure p_1

ε_2 = soil porosity factor after consolidation due to the weight of the embankment at a total pressure p_2.

The soil porosity factor varies with the change in load according to the following relation:

$$\varepsilon_2 = \varepsilon_0 - \frac{2.3 \log p_2}{A} \tag{146}$$

where ε_2 = porosity factor at pressure p_2

ε_0 = porosity factor at $p = 1$ kg/cm²

A = factor characterizing the soil compressibility and independent of loading.

Substituting the values of ε_1 and ε_2 into the equation of layer consolidation, the latter can be expressed as

$$\Delta = \frac{2.3H}{A(1 + \varepsilon_1)} \log \frac{p_2}{p_1} \tag{147}$$

The values of A and ε_0 for the compressibility curves of various soils are:

	ε_0	A
Fine sand and sandy loam	0.40-0.50	25-75
Silty soil	0.50-0.65	10-25
Loamy soil of medium density, argillaceous soil	Over 0.65	10-15
Sandy loam and loam containing more than 50% of silt; highly compressible loam and clay with interlayers of sand	0.70-0.85	5-10
Very highly compressible clay and macroporous soils	—	1-6

Peat, the decay factor of which is:

below 30%	0.35-0.45
40-50%	0.40-0.50
60-70%	0.55-0.75
80-90%	0.65-1.0
over 90%	0.75-1.5

Note: The lesser values of factor A correspond to the wetter peats.

If the bed soil consists of several layers differing according to consolidation characteristics or if the depth of the bed soil is such that it is necessary to take into consideration the attenuation of stresses with depth, the total settlement is calculated by summing up the settlements for each layer

$$\Delta = \sum_{i=1}^{i=n} \frac{2.3 \log \frac{p_{2_i}}{p_{1_i}}}{A_i (1+\varepsilon_{1_i})} h_i \tag{148}$$

When constructing an embankment on a peat base it is necessary to allow for the possibility of elastic vibration. These vibrations may cause the formation of cracks and the destruction of pavements, and also create additional resistance to vehicle motion.

It is considered that the elastic strain of a layer of peat left under an embankment should not exceed 0.5 cm. This means that the ratio of the filled layer of soil and of the soft ground left under the embankment for roads of classes IV and V must not exceed 1/3 in case of light metalled pavements, and 1/2 for intermediate and inferior pavements.

The consolidation of a saturated soil under the embankment may take a very long time. The rate of consolidation depends on the value of the soil coefficient of permeability, which may vary within a wide range. Thus, in the case of peats, depending on their composition and extent of decay, the coefficient of permeability may vary from 1×10^{-3} to 1×10^{-7} cm/sec.

The rate of settlement for structures on peat and silty bed soil can be determined by the theory of one-dimensional consolidation in relation to saturated soils. This theory in its simplified form examines the delayed compaction of saturated ground arising from diffusion of pore water under the pressure of the external load. It is assumed that the stresses in the individual compacted layers are uniform throughout the thickness of each layer, and the load is transmitted over a large area, the smallest of whose sides is 3-4 times greater than the thickness of a layer. The settlement after a time t from the commencement of load application, which is assumed to be instantaneous, is expressed by the following relation:

$$\Delta_t = \Delta_\infty \left[1 - \frac{8}{\pi^2} e^{-\frac{\pi^2 k t}{4 h_r^2 a (1+\varepsilon_{av}) \delta_w}} \right] \tag{149}$$

where Δ_∞ = ultimate settlement

$\pi = 3.14$

h_r = rated thickness of the consolidated layer. If the pressed out water is discharged through only one surface of the consolidated layer (in a sand bank, clayey swamp bed), the rated thickness h_r is equal to the full depth of the layer h. If the water can be discharged through two surfaces (sand bank and swamp bed), the rated thickness $h_r = h/2$

a = coefficient of consolidation which is determined from the equation of the straightened soil compression curve

$$\varepsilon_{av} = A - ap$$

δ_w = specific weight of water, assumed to be unity, and which is used in the formula to observe the required units

ε_{av} = average value of the soil porosity factor within the range of pressure variation from its value in the middle of the layer before application of the load to its value at the same place after application of the load

t = duration of load application

k = average value of the coefficient of permeability within the range of pressure variation from natural to p, cm/sec.

To facilitate the calculation one can make use of the existing auxiliary tables.

The rate of settlement may also be computed according to calculations based on laboratory tests of samples with undisturbed texture.

On the basis of the theory of earth consolidation, if the settlement of a sample h cm thick during t days is $q\%$ of its full settlement, then the time during which a similar relative settlement of a layer H cm thick will be achieved in reality, is determined from the ratio

$$T = t\left(\frac{H}{h}\right)^2 \tag{150}$$

If calculation shows that settlement of the embankment will not be completed during the period of construction prior to laying of the pavement, then this process can be accelerated by one of the following methods:

1. Increasing the excavated thickness of the peat layer (decreasing the thickness of the compressible layer).

2. Inserting a vertical sand drainage consisting of wells filled with coarse sand (Fig. 86). Experience in the use of vertical drains

shows that they can accelerate the settlement of the embankment from 20 to 25 times. In recent years vertical drains have become more popular in building practice. In Sweden in lieu of sand wells sheets of corrugated board are used with success; these are inserted into the ground by means of a special machine.

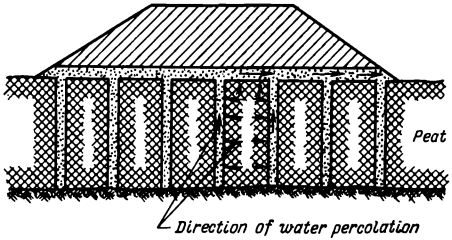


Fig. 86. Vertical drains provided to accelerate settlement of embankments on saturated bed soil

3. Draining the swamps.

4. "Overloading", which consists in the initial erection of a high but narrow embankment. This increases the embankment's specific pressure on the bed soil and accelerates settlement. Before laying the pavement the embankment is trimmed down to the formation level.

45. Stability of Side Slopes

The side slopes are the least stable part of roadbed embankments and cuttings, since the soil on the surface of the slopes is subject to the action of atmospheric precipitation and wind. With the disturbance of the moisture-content equilibrium the side slopes become prone to failure.

The experience gained during many years of earthwork jobs, e.g., the digging of trenches and channels, the building of hydraulic dykes and dams, of road and railway embankments, has enabled the gradient of slopes to be fixed empirically, thus ensuring the stability of embankments and cuttings that are not more than 10 metres higher or lower than ground level.

For highways the allowable gradients of embankment side slopes less than 1 m high are usually from 1 : 1.5 to 1 : 3, regardless of stability conditions, for convenience of operation of road machinery, and to provide for the possibility of vehicles running off the road.

Usually the accepted steepness of side slopes for embankments and cuttings in loose ground (sand and clayey soils) is 1 : 1.5.

The side slopes of cuttings on a hillside are maintained at a uniform gradient of 1 : 1.5 throughout their entire height.

For embankments filled with soils with a remoulded structure the 1 : 1.5 rate of rise of slope is limited to the following heights: in clayey and silty soil—6 m, in sandy loam and sandy soils—8 m.

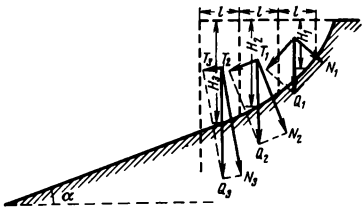


Fig. 87. Side slope shear plane

For high embankments the side slopes in the lower part are made less steep, with gradients of 1 : 1.75.

The side slopes of embankments which are subject to periodic flooding should have a maximum rate of rise of 1 : 2 within the limits of flooding and 0.5 m above the highest recorded or assumed water level. If the depth of flooding exceeds 6 m, the lower part of the side slope should be built with a gradient of 1 : 2.5. In reducing the gradient of the lower parts of the slopes the aim is to increase their stability and make their profile similar to the theoretical curvilinear one.

The side slopes of cuttings in stable soils may be steeper. The maximum gradient of slopes in gritty, gravelly and marl soils should be 1 : 1, for loessial soils in dry places it may be 10 : 1.

The stability of side slopes higher than 10 m, and also of slopes in saturated soils should be checked by calculation.

The limit contour of a stable slope in a cohesive soil, having an angle of internal friction φ and cohesion c, can be derived from the following considerations.

Consider an earth mass, limited above by a horizontal plane and on the sides by equidistant vertical planes. If the possible slip prism is separated by vertical planes we obtain a series of prisms of equal width. Upon examining the conditions of equilibrium of one of them (Fig. 87), let us assume that each separate prism remains in equilibrium independently of the adjacent ones, i.e., that the pressure and friction forces along the vertical faces do not exist.

The segregated prism tends to move along the shearing plane under the action of the tangent component of the force of gravity

$$T = Q \sin \alpha$$

The forces resisting shear are composed of forces of internal friction and cohesion, which are equal to

$$Q \cos \alpha \tan \varphi + c \frac{l}{\cos \alpha}$$

where c = cohesion
φ = angle of internal friction.

The conditions of limiting equilibrium corresponding to the equality of shearing and retaining forces, are expressed as

$$Q \sin \alpha = Q \cos \alpha \tan \varphi + c \frac{l}{\cos \alpha}$$

Dividing both parts of the equation by $Q \cos \alpha$ and taking into consideration that $Q = lh\delta$, where δ is the unit weight of soil, we obtain

$$\tan \alpha = \tan \varphi + \frac{c}{\delta h \cos^2 \alpha} \tag{151}$$

The above relation shows that the upper part of the side slopes in cohesive grounds can be stable with a vertical slope, but in the lower part long slopes should have a low gradient, with the angle of inclination approaching the angle of internal friction. This method was developed by Prof. N. N. Maslov for designing stable side slopes in heterogeneous soils. This method is based on the assumption that at the moment of shear a hydrostatic redistribution of pressure takes place in the soil (lateral pressure factor $\xi = 1$), and that the angle of slope stability for any soil is equal to the angle of shear ψ, the pressure on the soil being p, i.e.,

$$\tan \psi = \tan \varphi + \frac{c}{p} \tag{152}$$

The profile of a stable slope (Fig. 88) is derived as follows. The side slope is divided into a series of horizontal layers in accordance with its constituent beddings. At the foot of each of these the pressure due to its dead weight is determined

$$p_L = \Sigma \delta h \tag{153}$$

The values of the ermissible angles of shear are obtained by means of the relation

$$\tan \psi = \frac{1}{K} \left(\tan \varphi + \frac{c}{p} \right) \tag{154}$$

where K is the required safety factor. With $K = 1$ the contour of the slope having the limiting state of equilibrium is obtained.

The profile of the slope is determined in accordance with the obtained values of ψ, starting from the foot of the slope. The profile of the side slopes of high embankments and deep cuttings is determined by means of individual design.

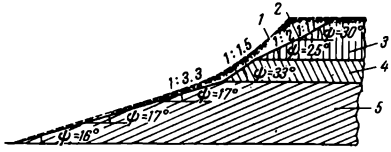


Fig. 88. Plotting profile of a stable slope:
1—natural slope; 2—design slope; 3—sandy loam; 4—loam; 5—clay

In order that the shape of the slope should conform to the contour of a stable slope a varying gradient is used for various horizontal sections or, alternatively, berms are introduced (Fig. 89). Berms reduce the velocity of runoff down the slope and prevent erosion.

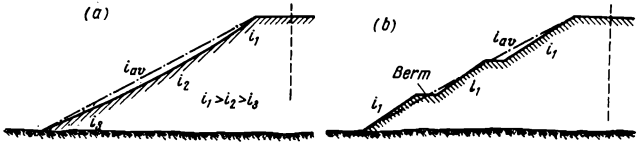


Fig. 89. Cross-sections of high embankments:
a—with varying slope gradient; b—with the use of berms

They also facilitate the maintenance and repair of slopes by providing places for inspection and the storage of materials.

As has been proved by observation, slip surfaces formed on unstable embankments may be considered as roughly cylindrical. Analysis of slope stability consists in assuming an arbitrary slip cylinder and then determining the corresponding stability numbers for the slip sections of the embankment. The slip cylinder usually passes through the toe of the slope if this slope is formed of uniform material.

If the base under the embankment is soft and has poor cohesion the area extending beyond the foot of the embankment will require investigation.

The method of cylindrical surfaces may be also used for stratified beddings, if the separate layers do not differ much from each other. The values of φ and c, which correspond to the properties of the soil intersected by the shear plane or to the direction of the shearing stress, are introduced for each layer separately into the expression of the stability number.

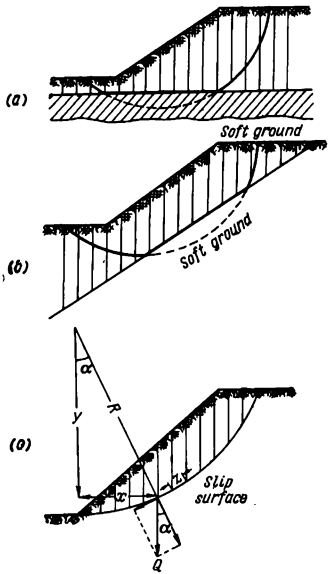


Fig. 90. Cylindrical slip surface:
a and b—corrections of shear plane contour to take account of heterogeneous soil beddings; c—diagram for determination of sliding slope stability number

If one of the intersected layers has a reduced resistance to shear, or if its resistance to shear in various directions is anisotropic, the possibility of the shear being confined within the limits of the soft layer should be considered (Fig. 90*a* and *b*).

The sliding surface which has the minimum stability number is determined when checking the stability of the slopes by the method of successive approximations, having assumed a series of such surfaces. When determining the side slope stability of filled ground the shear surfaces usually pass through the foot of the embankment. For cuttings, and also in the case of embankments on weak water-saturated bases, stability has to be checked also for slip surfaces extending to soil lying beneath the embankment.

To assess stability, each slip surface bounding a 1-m thick belt of the sliding mass is divided by parallel vertical sections into a series of prisms (Fig. 90*c*). The moments of forces, shifting and retaining each of the prisms, in relation to the axis of the slip surface, is determined by the expression

$$M_r = (Q \tan \varphi \cos \alpha + cl) R$$

$$M_{sh} = Q \sin \alpha R \qquad (155)$$

The stability number for the whole slope can be found from the ratio of the sum of the moments of the retaining and shifting forces:

$$K = \frac{\Sigma M_r}{\Sigma M_{sh}} = \frac{\sum\limits_{i=1}^{i=n} (Q \tan \varphi \cos \alpha + cl) R}{\sum\limits_{i=1}^{i=n} QR \sin \alpha} \tag{156}$$

However, according to Fig. 90, for each prism:

$$R \cos \alpha = y \text{ and } R \sin \alpha = x$$

Whence,

$$K = \frac{\sum\limits_{i=1}^{i=n} Qy + RcL}{\sum\limits_{i=1}^{i=n} Qx} \tag{157}$$

where $L = \Sigma l$ is the length of the slip surface after allowance is made for the fact that the failure of side slopes usually starts by the

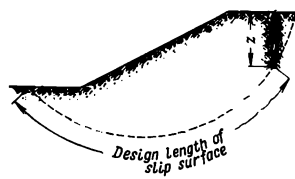


Fig. 91. Formation of crack in upper part of a sliding slope

formation of a crack in the upper part of the slope. This crack should be taken into consideration when selecting the layers and determining the length of the slip surface (Fig. 91).

The depth of the crack penetration will be

$$z = \frac{2c \tan \left(45° + \frac{\varphi}{2}\right)}{\delta} \tag{158}$$

In calculations it is often simplified to

$$z = \frac{H}{3}$$

where H is the height of the slope.

The degree of reliability of the slope stability check depends on how accurately the slip surface, for which the minimum value of the

stability number was determined, corresponds to the most dangerous surface arising within the slope

There is a number of methods for determining the centre of the most dangerous slip surfaces, based on lengthy experience of checking calculations. However, the degree of reliability of these methods has not been sufficiently evaluated, and no preference can be given to any of them. Therefore, the design organizations have to systematize

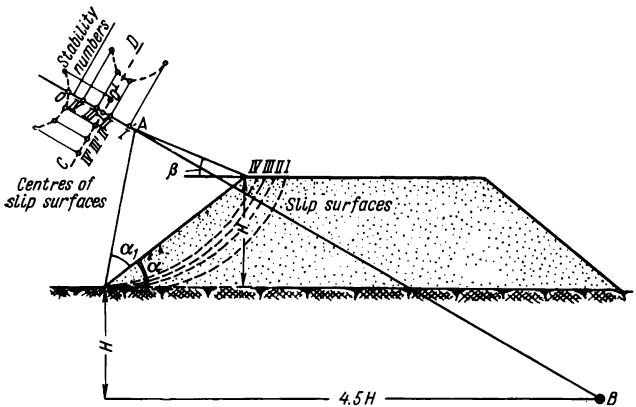


Fig. 92. Finding centres of slip surfaces having the minimum stability number

their calculation data in order to make the method used more accurate and to reduce the labour involved in determining the centres of critical slip surfaces.

In practice the most popular method is that of Fellenius, who has established that the centres of the slip surfaces corresponding to the minimum value of the stability number are situated next to a straight line *AB*. This line is obtained by plotting as shown in Fig. 92.

The values of angles α_1 and β, necessary for plotting the line *AB*, are given in Table 17 for various angles of slope.

To find the most dangerous slip surface one has to choose a series of centre positions (points *I*, *II*, *III* and *IV*) and to determine the stability number for each of them. The centre of the most dangerous slip surface is found as follows. Through a point on the straight line *AB*, corresponding to the minimum value of the stability number, a straight line *CD* is led at right angles. Several points are chosen on

it, for which the values of stability numbers are also found. The obtained minimum value of the stability number is assumed to be the design value.

TABLE 17

Side slope gradient	Angle of slope α	Angles, deg	
		α_1	β
1 : 0.58	60°	29	40
1 : 1	45°	28	37
1 : 1.5	33°40′	26	35
1 : 2	26°34′	25	35
1 : 3	18°26′	25	35
1 : 4	14°03′	25	36
1 : 5	11°19′	25	37

The saturation of the soil in the zone of contact with impervious layers by the incident rainfall is the main cause of spring and autumn slips in semi-stable slopes. The effect of water, penetrating into the roadbed during rainy seasons, snow thaw and floods, is to lower the stability number owing to the increase in weight of the soil upper layers moistened by rainfall. The coefficient of cohesion of the water-saturated soil decreases. In the lower part of the embankment which becomes saturated by seepage, the effects of flotation in water become apparent. In flooded embankments a hydrodynamic pressure D appears as a result of pore water drainage in the direction of the slope as the high water recedes—this is termed seepage or draw-down.

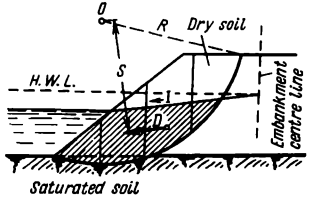


Fig. 93. Effect of saturation of a flood plain embankment on the side slope stability

The hydrodynamic pressure is assumed as being equal to the hydraulic gradient, i.e., the ratio of head loss to the length of percolation path. The value of the hydrodynamic pressure can be determined for the embankment fill experimentally by allowing water to percolate through a horizontal pipe filled with soil compacted to the same extent as in the embankment. The loss of head can be measured according to the water level in the vertical glass pipes—piezometers—attached to the pipe filled with soil.

When checking the side slopes of flood plain embankments, the calculation is made for the critical water table H.W.L., which corresponds to the minimum stability number and is determined by means of the graphical construction shown in Fig. 93.

The formula for determining the stability number of flood plain embankments is as follows:

$$K = \frac{\tan \varphi \Sigma N + c_1 L_1 + c_2 L_2}{D + \Sigma T} \tag{159}$$

where c_1 and L_1 = the cohesion and the length of the dry part of the slip surface

c_2 and L_2 = the same, but applying to the water-saturated part of the slip surface; the boundary between the dry and the saturated soil is assumed to be along the line of soil saturation or water table, the part of the embankment moistened only by capillary action being considered dry

ΣN = sum of the retaining forces

ΣT = sum of the shearing forces.

The unit weight of saturated soil can be determined according to the formula

$$\delta_s = \frac{(\delta_d - 1)(100 - n)}{100}$$

or

$$\delta_s = (\gamma - 1)(1 - n) \tag{160}$$

where δ_d = volume weight of dry soil

n = soil porosity, %

γ = soil specific weight.

The effect of flooding on flood plain embankments manifests itself differently according to the type of the soil. For sandy embankments, the soil of which has a high coefficient of permeability, only the effect of flotation in water has to be taken into consideration. Clayey flood plain embankments in which water penetration is low, do not become fully saturated during flooding and therefore these are treated as normal dry embankments. When the assessment is made of embankments filled with loam and sandy loam, the effect of all the above factors is considered. Runoff water, wind and wave impact cause the destruction of side slopes in embankments and in cuttings.

Simplified methods of stabilization are employed for cuttings, e.g., the sowing of grass and turfing. The stabilization of side slopes is a labour-consuming operation and so far has not been mechanized.

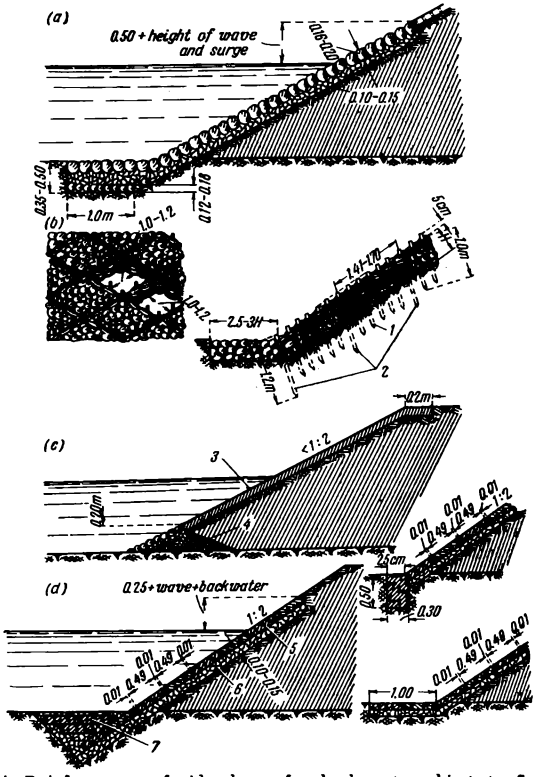


Fig. 94. Reinforcement of side slopes of embankments subject to flooding:

a—paving over a gritty or gravelly layer (with stones 16-20 cm high for water velocities up to 2.5-3.5 m/sec, with stones 25-35 cm high for velocities up to 3.5-5.5 m/sec); *b*—rock fill or paving in wattle casing (with rock fill and single-layer paving for velocities up to 3.5-4 m/sec, with double paving for velocities up to 3.5-5.5 m/sec); *c*—strengthening by laying fine-grained asphalt concrete (with velocities of 3.0-3.5 m/sec); *d*—by laying concrete slabs on a rubble or gravel base (with current velocity attaining 2 m/sec the slab thickness h=8 cm, with velocities of 2-3.5 m/sec h = 12 cm); *1*—stakes 5-10 cm thick, 1.0 m long; *2*—stakes 5-10 cm thick, 1.2 m long; *3*—fine-grained asphalt concrete 6-10 cm thick; *4*—rubble drainers provided every 10-20 m; *5*—concrete slabs; *6*—rubble or gravel; *7*—concrete slab apron

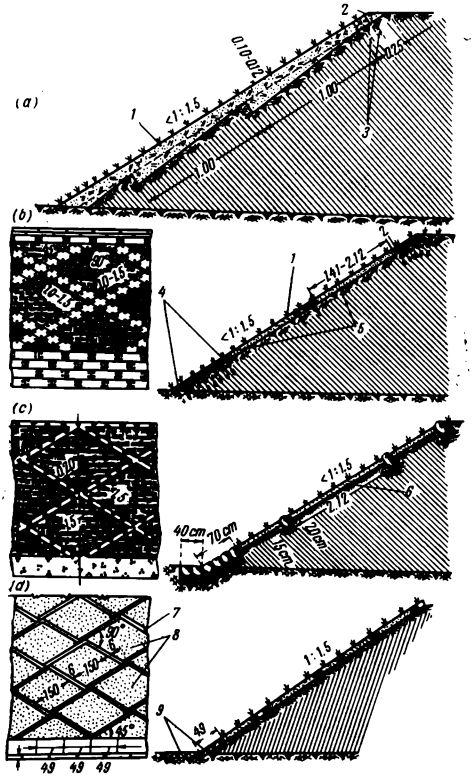


Fig. 95. Reinforcement of side slopes of cuttings and of nonflooded embankments:

a—strengthening of side slopes by seeding grass on a layer of vegetable soil 15-20 cm thick over sand or saline ground and 5-10 cm thick over other soils; *b*—strengthening of side slopes with tesselated turf and seeding of grass on a layer of vegetable soil 10-20 cm thick; *c*—reinforcement of side slopes with tesselated paving and grass on a layer of vegetable soil or with a layer of bituminous soil (with a 6-10 cm thick layer of fine-grained asphalt concrete the reinforcement can be used for flooded slopes at current velocities up to 3-3.5 m/sec); *d*—reinforcement of side slopes by tesselated laying of reinforced concrete blocks having a square or triangular cross-section and of seeded grass on a layer of vegetable soil; *1*—vegetable soil; *2*—layer of turf; *3*—wooden pegs; *4*—lower support consisting of 4 layers of turf; *5*—grass sown on a layer of vegetable soil; *6*—area sown with grass over a layer of vegetable soil, laying of fine-grained asphalt concrete; *7*—concrete blocks; *8*—seeded grass; *9*—concrete slabs 49 × 49 × 8 cm

Constructional practice has devised a number of methods for stabilizing the side slopes of embankments. These are shown in figures 94 and 95, and listed in Table 18.

TABLE 18

Type of reinforcement	Conditions of use	Maximum current velocities m/sec
Seeding with grass	Dry localities	0.6
Tesselated turf	Ditto	0.6
Continuous turf	Short-period flooding	0.9-1.4
Turf wall	Ditto	1.5-2.2
Stabilization by treatment with organic binding medium	Prolonged ponding	<5
Planting of shrubbery	Short-period flooding on flood plains	<3
Tesselated paving with turf	Prolonged ponding	0.9-1.4
Single-layer paving	Ditto	2-5
Rock fill	Ditto	3.0-4.5
Asphalt concrete	Ditto	3.0-3.5
Concrete slabs on rubble base	Ditto	6.5-10.0

CHAPTER 8

PAVEMENT DESIGN

46. Pavement Structural Layers

To ensure all-year-round operation of vehicular traffic on a road independent of weather conditions, the carriageway is covered with a pavement, which is a rigid or semi-rigid structure laid on the surface of the roadbed and resisting traffic stresses and climatic factors.

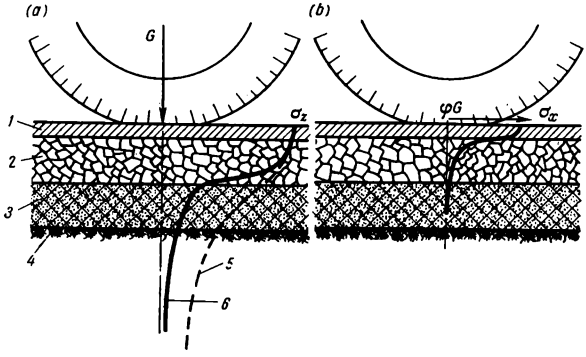


Fig. 96. Traffic stress diagram for a multilayer pavement:
a—diagram of vertical stresses σ_z; *b*—diagram of horizontal stresses δ_x; *1*—wearing course; *2*—base course; *3*—sub-base; *4*—sub-grade; *5*—homogeneous soil; *6*—pavement

The stresses induced in the pavement by motor vehicle wheels attenuate with the depth (Fig. 96). This enables the pavement to be designed in the form of a *multilayer structure*, employing materials whose strengths vary for each layer and are determined in accordance with the magnitude of the acting forces. The pavement consists of the following layers:

1. *Surfacing* is the upper, and most rigid, layer of the pavement. It is comparatively thin, but resists well the abrasion and the impacts caused by the wheels, and also the effect of weather conditions. Usually, the surfacing is the most expensive part of the pavement and, therefore, is laid to the minimum admissible thickness. The surfacing provides the required road service qualities (surface smooth-

ness, high coefficient of adhesion). Surfacing usually comprises two coats or courses—the *base course*, on which depend the basic qualities of the surfacing, and a *wearing course*, which is not regarded in calculations and which is periodically renewed as it wears out. When the surfacings are made of weak materials, which are subject to appreciable wear, a special wearing course made of strong stone material treated with organic binders is necessary, which may be periodically renewed in the course of road operation.

If the surfacing is not sufficiently impervious to water and may be destroyed during freezing or drying out in hot arid weather conditions, it is covered with a thin *protective* or *sealing coat* by surface treatment with a binder and a filling of fine sand. Surface treatment is also used for increasing the roughness of polished surfacings.

2. Below the surfacing base coat is the *pavement base*, a strong bearing layer of stony material or stone with a binding matrix. This layer is designed to distribute the individual wheel-loads over the roadbed or sub-base.

The pavement base is not subject to the direct action of automobile wheels. Therefore, materials of a lesser strength than those used for the surfacing or the wearing course can be employed in its construction.

When the base is protected from the action of surface water—in the case of an impervious surfacing—it may become saturated by water drawn upwards from the roadbed during winter frost penetration. For this reason, in the northern regions materials used for base construction have to satisfy certain requirements concerning frost resistance.

3. The sub-base is a layer of earth or stone materials, resistant to moisture, inserted when necessary between the pavement base and the roadbed to reduce the required thickness of the pavement base. The sub-base is made of gravel, slag, soil treated with binding agents, sand, etc.

On sections where the roadbed comprises silty, loamy and clayey soils, inside which winter moisture accumulation may occur, a sub-base of porous materials is introduced. This consists of a sand or gravel layer which drains away excess water from the upper layers of the roadbed, drains the pavement structure and increases the bearing strength of the roadbed. It is termed a *drainage* or *anti-frost heave course*.

If the roadbed is composed of stable, impervious sand, sandy loam or gravel soils, a sub-base is not necessary.

4. The *subgrade* comprises the thoroughly compacted upper layers of the roadbed, upon which are laid the layers of the pavement. The subgrade receives all the distributed pressure of traffic loads and,

therefore, is a very important element of the pavement structure. The stability of road pavements can be ensured only on a heterogeneous, well compacted roadbed with adequate drainage. The increase of roadbed soil resistance to external loading, its drainage and the uniformity of water conditions are the best means for ensuring pavement stability and reducing its cost. No increase in the thickness of the pavement base can guarantee the strength of a pavement laid on a weak bed soil.

47. Main Types of Pavement

To permit automobile traffic to travel along a road at any time of the year at high speeds and with economic fuel consumption, the road pavement must be of an adequate rigidity, uniformity and resistance to wear. These requirements can be satisfied by means of various combinations of pavement structural layers consisting of different road-building materials. The pavement service qualities, i.e , permissible speed and traffic comfort, are determined mainly by the nature of the surfacings, which can be divided into the following basic structural types, given in consecutive order of their development.

Cement concrete and asphalt surfacings (Fig. 97*a* and *b*). These surfacings are of high rigidity and of high resistance to loading.

The stone aggregate is thoroughly graded, so that the interstices between large particles are filled with smaller chips, and the material as a whole has a minimum porosity (maximum density). Cohesion is provided by the use of cement and organic binders.

In contrast to asphalt surfacing, cement concrete surfacing has a very considerable inherent strength and temperature stability. These surfacings usually consist of separate concrete slabs, measuring 3-4 m by 6-7 m. The slabs are separated from each other by joints which are necessary to allow for changes in length owing to temperature fluctuations There are expansion joints which contract when the slab length increases, and contraction joints which expand when the slabs shrink. Inserted into the joints are steel bars called dowels which provide for the possibility of small changes in slab length but which transmit vertical loads from one slab to the other, and, to a lesser degree, flexural moments.

The component parts of cement concrete pavements, which are closely connected with the features of their construction, are described in detail in special courses on highway construction.

Apart from sand, stone dust (mineral powder) is introduced into the asphalt concrete, which enters into physical and chemical reactions with the organic binding agents, resulting in the surfacing becoming more resistant to temperature change.

The asphalt concrete surfacing is flexible and should, therefore, be laid over a solid stone base (flexible pavement).

Bituminous macadam—broken-stone and gravel surfacing treated with organic binders. Owing to the adhesive properties of the

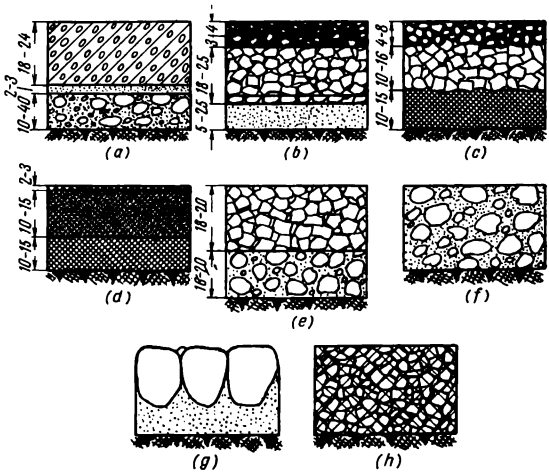


Fig. 97. Structures of main types of pavement:

a—cement concrete pavement on a gravel base; *b*—double layer of asphalt concrete on a rubble base and a sand sub-base; *c*—stabilized broken stone surfacing, laid on an earth base treated with binders; *d*—double-layer pavement of soil-cement with surface treatment; *e*—broken stone pavement on a gravel base; *f*—gravel pavement; *g*—cobblestone pavement; *h*—earth pavement stabilized with rubble additions (soil-aggregate mixture)

binders, this surfacing is highly resistant to the destructive action of traffic. Such a pavement is impervious to water.

The differences in the methods of binder introduction in the process of construction create the fundamental structural characteristics of the surfacings obtained.

A. Mixing on the road site or in special plants provides for good coating of the chippings by the binder. The amount of binder used is less with this method than when using the method of impregnation. The mixing method together with the proper selection of stone material grades makes possible the provision of stronger surfacings.

The positive mixing makes possible the use of chippings graded in such a way as to form a solid matrix, the density of which approaches the optimum value.

B. Impregnation (Fig. 97*c*) is the introduction of the binder into the surfacing by means of pressure-spraying over the surface of a lightly compacted layer of uniform chippings.

After the penetration of all of the bitumen into the interstices of the aggregate, the surface of the pavement is covered with fine chippings and compacted by rolling. The stability of surfacings of the impregnation type is ensured mainly by the wedging action of the chippings, which takes place during the rolling process. Among the shortcomings of this process is the comparatively high consumption of binder per unit area. The bitumen, percolating along the interstices between the chippings, does not penetrate between the points of their contact, where its action would be most effective, but forms interspatial clots.

C. Surface dressing (Fig. 97*d*) is a thin protective coat created on the surface of the pavement by spraying a layer of bitumen over it which is then covered with very fine chippings. Depending on the number of bitumen applications a single or double surface treatment may be applied. Surface dressing increases the resistance to wear and makes the surfacing impervious thus enabling the pavement surface to remain dry during wet seasons and so retain a high modulus of shear. However, account is not to be taken of this effect when calculating pavement thickness. When chippings of very hard rock are used for surface dressing, the coefficient of adhesion between tyre and road increases and appreciably improves the safety of traffic.

Sometimes the surface dressing is considered not as a separate type of surfacing, but as a wearing course laid to reduce the wear of the main surfacing, e.g., asphalt concrete, and for rendering it impervious. However, when surface dressing is applied to low-quality roads, the improvement of their service properties and increase of their strength in autumn and spring are so extensive that surface dressing may be considered as an independent type of surfacing.

Broken-stone surfacings (Fig. 97*e*) and bases made of uniform size chippings (macadam). The strength of broken-stone surfacings is provided by the wedging action which takes place during rolling. The major factor determining the stability of the surfacing is the friction developed between chippings, also the cementing action of the stone powder formed by abrasion of the chippings during rolling. The abrasion of the edges and the crushing of the stone, in addition to the penetration of mud deposited on the surface during use of the road give rise to the appearance of sandy, silty and clayey particles within the interstices and hence to the loss of cohesion by the surfacing, especially during wet seasons.

Broken-stone surfacings have a low resistance to wear under automobile traffic, since the tangential stresses of pneumatic tyres destroy the efficiency of packing. Consequently, such pavements are used as an independent type of surfacing only when the traffic intensity is low. More often they are used to provide the road with a base laid beneath a surfacing treated with organic binders.

Surfacings of natural gravel or artificially graded gravel mixtures (Fig. 97*f*). The strength of the material is provided by grading as closely as possible to the optimum mixture, keeping the interstices between big particles filled with finer ones so that the material, as a whole, has the minimum porosity. Cohesion is achieved by introducing fine mud and clay particles into the mixture. In humid seasons of the year the strength of the surfacing may be reduced owing to the decrease of cohesion.

The gravel road is the cheapest form of road and the simplest from the construction point of view. It has high strength and stability when it does not contain an overlarge quantity of fine fractions, which make the mixture plastic in wet conditions. Pavements of local weak materials and of industrial waste products (blast furnace slag, cinders, bog iron ore, burnt shale from coal mine dumps) are constructed in a manner similar to the gravel type roads.

Pavings (Fig. 97*g*) are surfaces and bases made of individual natural or artificial stones placed close to each other, usually forming an interlocking pattern.

Manufactured pavings, made from wood blocks or clinker, provide a smooth surface. Pavings of coarse hewn or boulder stone (cobblestone pavement) are used on roads of classes I-III as temporary surfacings or as a base for higher quality surfacing; and for roads of classes IV and V as the surfacing proper.

Somewhat similar to cobblestone pavings are pitched or penned bases, i.e., stones placed with their large bases (bedding) downwards. The subsequent filling and wedging of the interstices (blinding) with rubble make for the construction of an almost monolithic base.

The main drawback of cobblestone paving and pitched bases, from the point of view of work organization, is the necessity of laying them (the paving) by hand, which does not harmonize with the requirements of modern, mechanized high-speed construction. For this reason the use of pavings for arterial highways is constantly decreasing. However, for local roads they may be of great importance for a number of years to come.

Earth road pavements stabilized with granulometric additions (Fig. 97*h*) consist of local soils, whose resistance to wetting, if clay, or insufficient cohesion, if sand, are materially improved by the addition of other soils, which have the equalities lacking in the local soil.

The addition of sand, gravel and other granular materials to clay increases its resistance to external loading in wet conditions. The addition of hard material increases the strength of the soil and decreases the wear of the pavement. Sand is given stability and cohesion by the introduction of loam or clay.

Natural earth roads actually have no pavement. The carriageway comprises the upper layers of the natural ground compacted by traffic. These roads can only serve for carrying traffic of low intensity in dry seasons of the year. The main factor limiting the traffic intensity on earth roads in summer is the formation of dust.

During rainy periods earth roads will become slippery. The adhesion of pneumatic tyres to the road surface falls sharply and the vehicle wheels start spinning. Under persistently wet conditions deep ruts are formed on these roads.

Depending on the riding quality road pavements are classified as high-quality, intermediate and inferior. When classifying road

TABLE 19

Types of pavements	Main kinds of pavements	Maximum traffic intensity for two lanes, standard vehicles per day
High-quality heavy-duty	1. Cement concrete	> 3,000
	2. Hot and warm asphalt concrete	> 3,000
	3. Pavements of strong broken-stone graded materials processed in mixers with viscous bitumens or tars	1,500
	4. Stone block or mosaic pavings on stone or concrete base course	3,000
High-quality light-duty	1. Cold asphalt concrete pavements	1,500-3,000
	2. Pavements of crushed stone and gravel materials stabilized with viscous organic binders	1,500-3,000
	3. Ditto, with liquid bitumens	1,500
	4. Ditto, of soil processed in a plant with viscous bitumens	1,500
Intermediate	1. Broken-stone pavements of natural stone materials, gravel or slags (with surface finishing)	1,000
	2. Of soils and local weak aggregates stabilized with liquid organic binders	500
	3. Cobblestone and broken-stone pavements	500
Inferior	1. Soil pavements stabilized with various local materials	100

pavements the decisive factors are the permissible traffic speed and the rate of strain accumulation in them. The classification of road pavements is given in Table 19.

Bases beneath heavy-duty surfacings must maintain the requisite strength throughout the year, without showing any decrease during the wet season.

Bases under the high-quality surfacings shown in Table 19 may be made of the following types, depending on the required strength of the pavement and the availability of local building materials: broken-stone, gravel, blast-furnace slag, cinder and other local industrial waste materials, of the soil used for the roadbed treated with binders, i.e., bitumen, cement or lime.

The intermediate and inferior types of pavements are laid directly on the bed soil, with the exception of broken-stone pavements, which should be laid on a base of soil treated with binders, or of slag or other local materials.

In regions where there is no spring moistening of the roadbed soil and under favourable ground conditions, pavement structural layers of stone may be laid directly on the roadbed without the use of a sand base.

The separate types of pavements conform to road classification in accordance with the traffic intensity as indicated in Table 19*a*.

TABLE 19*a*

Average daily traffic	Recommended types of road pavements
$>6{,}000$	High-quality heavy-duty
$>3{,}000$	High-quality heavy- and light-duty
From 3,000 to 1,000	High-quality heavy- and light-duty, intermediate
From 1,000 to 200	High-quality light-duty, intermediate, inferior
$\leqslant 200$	Inferior

48. Choice of Pavement Type

As was shown in tables 19 and 19*a*, various types of road pavement construction may be used for the same traffic intensity. In the planning stage the choice should be made from several possible types and the most appropriate pavement should be selected, taking account of the traffic requirements, local natural conditions, availability of local building materials, and of the facilities offered for organizing construction work. Various types of equal strength should be

compared, with their dimensions preliminarily checked by the relevant calculations.

When making the final choice of pavement construction from several possible versions, as determined by local conditions, preference should be given to the most economical solution, taking into account the cost of construction as well as the cost of maintenance, repair and vehicle operation.

The most desirable type of pavement is the one which provides during the assumed repayment period the minimum cost per ton-kilometre, as computed from the sum of the vehicle operation costs and the expenditure for construction, operation and repair of the highway, i.e., the *vehicle and road components of the total transportation costs.*

The *vehicle component* of the transportation costs comprises outlay for fuel, lubricants and tyres, drivers' wages, vehicle repair and maintenance costs. also the depreciation in the value of vehicles. An appreciable part of these costs depends on the type and state of the pavement (Table 20).

TABLE 20

Type of pavement	Rolling resistance factor	Relative operating indices			
		Mean speed	Fuel consumption	Mileage between overhauls	Transportation costs
Asphalt and cement concrete	0.015	1	1	1	1
Broken-stone pavement with organic binders	0.025	0.95	1.05	0.94	1.1-1.3
Broken-stone and gravel pavement	0.035	0.7-0.8	1.1	0.83	1.2-1.6
Cobblestone paving	0.05	0.65	1.3	0.83	1.4-1.8
Shaped earth road	0.05-0.06	0.4	1.6-1.8	0.51	1.7-2.0

Note: The lower values of the transportation costs relate to a highly organized system of vehicle operation, the higher ones to average conditions of vehicle fleet operation.

Since transportation costs are usually related to one ton-kilometre, the use of heavy and combination vehicles reduces the vehicle component of these costs.

The *road component* of transportation costs is composed of expenditure for road construction, running repairs and major overhaul, and road maintenance, related to one ton-kilometre.

The result of these calculations shows that the road component does not exceed 10 to 15% of the total transportation costs (Fig. 98). With an increase in the length of haul and vehicle capacity the rela-

tive part of the road component will become greater, although the total value of the transportation costs will decrease. Since the increase of the traffic speed on the road has an appreciable bearing on the reduction in transport costs, the design and maintenance of highways should be aimed at increasing the permissible traffic speed.

The final choice of the type of pavement is made by comparing the periods necessary for repayment of the capital cost of construction with the economy in the relevant operation costs.

The calculation is made as follows:

(a) Several patterns of pavement structure are designed which will satisfy the traffic requirements and the conditions of work organization.

(b) For each version constructional and annual overhaul and transport costs are determined.

The design construction cost of the various types of pavement is assessed in conformity with similar completed projects, giving due consideration to possible differences in material delivery. If the required data are not available, an estimate of expenditure is made.

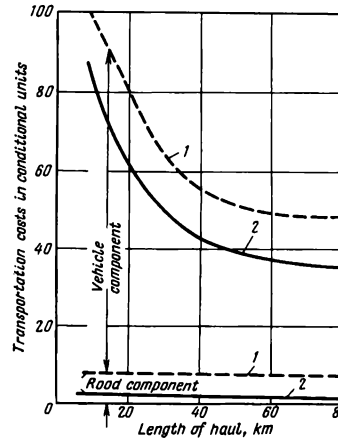


Fig. 98. Relation between vehicle and road components of transportation costs: 1—broken-stone pavement; 2—asphalt concrete pavement

The traffic operation costs are dependent on the traffic intensity N and are composed of the running, overhaul and major repair costs. Thus the traffic operation costs per 1 km of road per year may be expressed as follows:

$$R = 365N\Sigma n_i a_i k_i + N(r + b) \qquad (161)$$

where a_i = cost of 1 t-km of transportation on vehicles of various types

n_i = vehicle capacity

k_i = relative number of vehicles of various types in the traffic stream

r = road operation costs

b = cost of road repair.

Usually when comparing the construction and operation expenses for various types of pavements it will appear that with smaller construction expenses the operation costs increase, i.e., if $C_1 < C_2$ then $R_1 > R_2$.

The period during which the additional expenses for building a higher quality pavement will be justified, is determined by the formula

$$t = \frac{C_2 - C_1}{R_1 - R_2} \tag{162}$$

Suitable periods for writing off the capital cost of construction of the various types of road pavements, which may be used as a guide when comparing various pavement versions, are given in Table 21.

TABLE 21

Traffic intensity, vehicles per day	Type of pavement			
	Heavy-duty high-quality	Light-duty high-quality	Intermediate	Inferior
	Number of years required for repayment of cost for building various types of pavements			
5,000-3,000	6-8	—	—	—
3,000-1,000	6-8	6-8	—	—
1,000-200	—	10-12	10-15	—
$\leqslant 200$	—	—	15-20	20

49. General Principles of Pavement Analysis and Design

The pavement is the most expensive part of a highway, the cost of its construction being from 40 to 60% of the total construction expenditure. At the same time the pavement operates under more arduous working conditions than other road structures since it is subject to the direct action of traffic loads and of natural factors. Therefore, the selection of the pavement structure has to receive special attention, requiring the combination of adequate strength and the application of every reasonable means of reducing construction costs.

The last problem is the most important, since in many regions there is a scarcity of local stone materials. Road building is associated in these regions with the employment of imported stone, which sometimes has to be brought by rail for distances of hundreds of kilometres. In these conditions it is important to build road pavements

with a minimum admissible safety factor, which will just satisfy the strength requirements and those of resistance to atmospheric agents.

The planning of road pavement structures consists of two consecutive stages—design and analysis, which are interrelated and should not be opposed to each other. The substitution of one for the other cannot guarantee a stable, economic and operationally efficient pavement.

Pavement design consists in determining an expedient arrangement of pavement layers and selecting the material for their construction, having regard to local resources and the organization of work. Pavement design is the most creative part of the projecting work. It should be based on a clear understanding of road pavement stress and strain processes, on the use of experience gained from the operation of various types of pavements in different climatic conditions, and on an appreciation of the influence of natural phenomena.

Road pavement design has to take into account the peculiarities of pavement construction, preference being given to the patterns requiring the least material resources and not requiring the use of hand labour.

In the process of designing the thickness of the individual pavement layers is determined not so much by the pavement strength, as by other factors such as the discharge of water and the resistance to wear. Alternatively they may be selected to be of the minimum thickness because of their high cost.

Analysis of a pavement structure consists in justifying the necessary thickness of individual layers and of the pavement as a whole. It must provide for equal strength of various pavement types and for their suitability to given traffic conditions.

The pavement behaviour is so complicated, that no one of the existing road pavement analysis methods is capable of encompassing it completely. Hence one always has to admit a safety factor to allow for circumstances not envisaged by the scheme used for the analysis.

One of the main requirements when selecting pavement construction is the assessment in each case of the traffic intensity and of local soil, hydrologic and climatic conditions, which influence pavement service. Thus, for instance, broken-stone and gravel pavements which have not been treated with organic binders will provide better service in humid, temperate climates but will disintegrate rather rapidly under the conditions of a southern, arid climate. In regions having a humid maritime climate, where the roads often become slippery, it is necessary to increase the roughness of surfacing. In regions with excessive rainfall it is necessary to provide for special drainage layers.

The climatic conditions also influence the choice of pavement types since they may limit the duration of the building seasons, e.g., for works connected with the use of organic binders, etc. In arid regions the use of cement concrete pavements is complicated by the difficulty of supplying water to the site and arranging for proper curing.

One of the main requirements for the choice of the road pavement structure is the ability to make the maximum use of local building materials. It is well known that delivery costs constitute an important part of road building expenditure. Therefore any reduction in the haulage distance of road building materials substantially reduces the cost of the whole construction work. In regions where there are no cheap local stone materials, one should attempt to utilize local soils which may be treated with binders.

Structures should be preferred which are the simplest to construct and which permit the manufacture of prefabricated parts, as well as allow full mechanization of works on the site. All this facilitates the organization of a road building system employing continuous high-speed methods. The number of pavement structural layers should not be increased without obvious necessity, since this usually complicates the technological process and thus increases the cost of construction.

When designing pavements which are to be strengthened later as the traffic intensity increases, their strength is usually provided by the layers which will subsequently be used as the pavement base. At first thin surfacings are laid over these layers, or wearing courses which may be continually restored.

Materials used for the road pavement structure are arranged according to decreasing strength, corresponding to the attenuation of stresses with depth in the case of superimposed loads. It is most expedient

	Minimum thickness of layer, cm
Hot or cold asphalt concrete:	
(a) single-layer	4
(b) double-layer	7
Cold asphalt concrete and tar concrete	2
Broken-stone and gravel materials and soils stabilized with binders in special plants	4
Broken-stone stabilized by means of the penetration method	4-7
Broken-stone and gravel materials stabilized with binders by mixing in place	5
Soils stabilized with organic binders by mixing in place	6
Soils stabilized with cement or lime	10
Broken-stone and gravel materials:	
(a) on sand base course	15
(b) on strong base course of stone or stabilized soil	8-10

to maintain the ratio of the moduli of strain of adjacent layers within the range of 1.5-3.

The strength of road pavements should be developed by using as far as possible cheap local materials (gravel, soil treated with binding agents) while imported materials should be used in layers of minimum thickness, forming them into a solid constructional layer which will ensure reliable service of the pavement. The thickness of separate pavement structural layers should not be less than specific minimum values.

50. Pavement Loading

The pressure of automobile wheels on the pavement is the main load taken as the basis for the analysis of road pavements.

Modern vehicles are fitted with pneumatic tyres with an inflation pressure between 1.5 and 7 kg/cm². There are low-pressure tyres with an inflation pressure of 1.75-5.5 kg/cm² and high-pressure tyres of 5-7 kg/cm².

The wheel load is transmitted to the road surface through an area equal to

$$\omega = \frac{G}{kp_0} \tag{163}$$

where G = wheel pressure on the surfacing, kg

p_0 = tyre inflation pressure, kg/cm²

k = factor allowing for the influence of tyre side wall rigidity, equal to 1.0-1.3 for various types of tyres. For the analysis of pavements the mean value is taken, i.e., 1.1.

In reality, the mean pressure applied to the surface in contact with the tyre is somewhat higher, since the tyre contact is made not over the whole area, but through the projections of its tread. However, this has no effect on the strength of the pavement because of the distribution of stress within the pavement, but influences only the rate of wear of the surface.

The very clear picture of pressure transmission from a stationary vehicle on a surfacing alters substantially when the vehicle starts to move. There are always irregularities on the road surface, which cause vehicle vibration during its motion, inducing periodic variations in the wheel pressure on the surfacing.

In traffic conditions the specific pressure of a wheel on the surfacing also increases. This happens because of the influence of a number of factors:

(1) the heating of the tyre and the consequent increase of its inflation pressure;

(2) the increase of tyre rigidity owing to the effect of centrifugal force which extends the tyre;

(3) the short duration of contact of each tyre area with the surfacing, as a result of which the tyre has no time to deform to the extent corresponding to the static application of the actual load, i.e., it appears to become more rigid.

The theoretical analysis of vehicle wheel pressure on a road has been carried out by a number of specialists. However, the complexity of the theoretical analysis and the random disposition of irregularities on the road surface, which can be expressed only conditionally in a mathematical equation, give preference to the experimental methods of measuring vehicle pressure on a road.

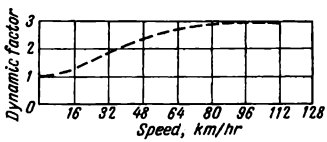


Fig. 99. Results of tests measuring wheel pressure on pavement during movement over irregular road surface

The use of special instruments—accelerometers—for measuring vertical accelerations of a vehicle at the moment of passing an irregularity, proved that with the growth of traffic speed the wheel pressure on the road surfacing also increases. This rise in pressure is due to the instantaneous increase of impact stress when the wheel hits an irregularity at a greater speed, and to the increase of tyre rigidity at high speed.

Figure 99 shows the results of measuring the wheel pressure on the surfacing with multiple test passages of vehicles over rough stretches of high-quality pavements, the deformations of which approached the maximum ones occurring on the road in normal working. The tests led to the conclusion that with speeds up to 80 km/hr the pressure on the surfacing increases almost proportionally to the speed, and beyond that speed it remains practically constant.

However, simultaneously with the increase of pressure on the surfacing, as the vehicle speed increases, the reverse processes take place in the road pavement base. Investigations of various pavements by means of test loads show that the full amount of pavement flexure, corresponding to the applied load, is attained only after a few minutes interval. A sudden impact of a rolling wheel on the road, owing to a retarded strain and inertia resistance, induces a lesser pavement

flexure than a static load. For a subgrade this is equivalent to a decrease of the applied pressure.

Upon measuring the alteration of stresses in the bedding courses under a road pavement, it was established that with a smooth road surface the stress in the bed soil due to a moving load is less than that due to a static one (Fig. 100). With an uneven surface the dynamic factor for the subgrade exceeds unity, but is less than the factor

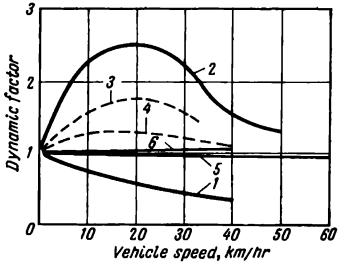


Fig. 100. Alteration of stresses in road pavement subgrade with speed of vehicle:
1—earth road; 2—cobblestone pavement; 3—broken-stone pavement stabilized with binders having pot-holes; 4—ditto, with smooth surface; 5—smooth broken-stone pavement; with surface treatment; 6—asphalt concrete pavement

measured by the direct impact of the wheel on the road surface. The dynamic factor increases with the increase of surfacing irregularity.

Since the road maintenance and overhaul service should maintain the pavement surface in a smooth condition, account is not taken of the dynamic factor during the pavement design stage. Indirectly its effect is expressed in the coefficient of correction for the recurrence of vehicle load application.

In the U.S.S.R. the design load on highways is assumed to be in the form of a reference or standard vehicle in accordance with the standards for mobile vertical loads used for the design of highway structures (standard H-106-53, diagram H-13). The rated specific pressure of a wheel of this vehicle is 5 kg/cm², and the circle diameter equivalent to the wheel imprint area is $D = 34$ cm. The actual traffic intensity on a road of various types of vehicles is reduced to the equivalent flow of the standard vehicles by using the methods described below in Sec. 52.

51. Strength of Flexible Pavements

Pavements are called flexible when their resistance to flexure is small. They comprise practically all the types of pavement except cement concrete, asphalt surfacings and stone pavings laid on a cement concrete base.

The strength of flexible road pavements is caused mainly by the resistance of the sub-soil to compression. The part played by a flexible pavement is largely that of decreasing the specific stresses transmitted to the bed soil. The reduction in the quantity of surface water

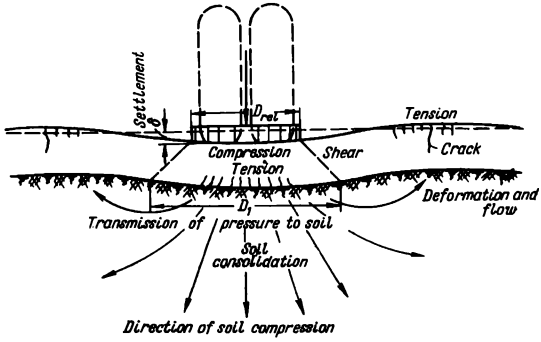


Fig. 101. Road pavement strains

percolating into the subgrade also has significance. The action on a flexible pavement of various loads transmitted through a standard circular plate can be represented in the form of a curve of pavement flexure versus load which is similar to that shown in Fig. 82 above. With applied loads that are small in comparison with the load which destroys the pavement, strains are small and there is complete elastic recovery upon removal of the load. Recovery is retarded, however, and requires prolonged time, hence these strains are known as elasto-plastic deformations. With a heavy loading permanent plastic deformations of the sub-base occur, causing destruction of the pavement when they attain a "critical" value.

The occurrence of permanent plastic deformation in road pavements is the result of the development of a number of processes (Fig. 101).

1. The soil under the load compresses causing flexure of the road pavement along a certain curvilinear surface forming the so-called flexure bowl.

The greater the thickness and the rigidity of the road pavement, the larger is the area over which the pressure of an external load is distributed and, therefore, the smaller are the stresses compressing the soil.

2. In the pavement immediately beneath a wheel load compression of the surfacing material takes place, while in the lower part tension occurs. If the surfacing has no resistance to tension, cracks appear in it. Along the perimeter of the wheel-to-surface contact area shear stresses are induced which, with heavy loads, cause failure of the pavement situated under the wheel, the sheared surface taking the form of a truncated cone. The relative destructive influence of each of the above strains on the road pavement is not yet clear. The thinner the pavement and the lesser its relative rigidity in relation to that of the bed soil, the more frequent is the occurrence of failure.

Depending on the requirements which the road is to comply with, the design of the pavement layers can be based on the conditions for attaining given magnitudes of strain. Experts consider that a pavement subject to the design load should operate within the elastic strain stage, occurring at the time of the year when the soil has its minimum strength. Compliance with this requirement leads to an increase of pavement thickness.

The process of calculating flexible road pavements must allow for the possibility of accumulation of the permissible strain during the season of spring or autumn bed soil saturation.

The investigations of flexible road pavements by test loads have shown that the magnitude of critical flexure at which the pavement fails varies according to the following empirical relation:

$$\lambda = \frac{\Delta}{D} = 0.1\,\frac{\alpha}{\pi}\arctan\frac{h}{D}\sqrt[2.5]{\frac{E_p}{E_s}} \qquad (164)$$

where E_p = pavement modulus of strain, kg/cm²
E_s = soil modulus of strain, kg/cm²
h = pavement thickness, cm
D = diameter of the circular area, through which the load is transmitted onto the pavement, cm
α = factor which depends on the type of the pavement; for asphalt $\alpha = 1$; for broken-stone and gravel pavements treated with organic binders $\alpha = 1.1$; for bases of soil stabilized with binders $\alpha = 1.25$; for broken-stone and gravel pavements and stone pavings $\alpha = 1.1$-1.2.

The magnitudes of flexural deformations calculated according to this formula range from 0.035 to 0.06 of the area diameter D.

It is recommended to assume for calculation purposes a mean value of λ related to the type of pavement as shown in Table 22.

TABLE 22

Road class	Type of pavement	Name	λ
I-II-III	High-quality heavy-duty	Asphalt concrete, broken-stone precoated with bitumen in a plant	0.35
III-IV	High-quality light-duty	Broken-stone and gravel stabilized with binders on site	0.4
IV-V	Intermediate	Broken-stone, cobblestone	0.5
		Soil cement	0.4
		Gravel, soil stabilized with bitumen	0.6

The bed soil plays an important part in ensuring the strength of flexible road pavements, since it carries all the vehicle loads which are transmitted to it through the pavement. The strength and smoothness of the road pavement depends on the extent of bed soil deformation and on its resistance to external loads. The strongest heavy-duty pavement will rapidly lose its smoothness and become uncomfortable to ride on, if it is constructed on a loose and heterogeneous bed soil.

Soil is a flexible plastic material, the deformation of which is not directly proportional to load. Therefore, though all the calculations of bed soil stability are based on the assumption of *linearity between stress and strain* (theory of elasticity), the values of soil *strain characteristics* should be determined in accordance with the observed magnitude of deformations. Such a method is so far the only possible one for practical application, since to date the theory of nonlinearly strained materials has been insufficiently developed.

The resistance of soils to external loads is evaluated by their modulus of strain.

If at a certain depth in the bed soil there are compressive stresses σ which cause a reduction δ in the thickness of a thin layer h, then the modulus of strain will be

$$E = \frac{\sigma h}{\delta} \tag{165}$$

The moduli of strain are usually determined experimentally, by forcing into the soil a solid cylindrical plunger. The summing up of strains in a mass of soil due to stresses attenuating with the depth, leads in this case to the relation

$$E = \frac{\omega p D}{\Delta} \tag{166}$$

where p = specific pressure on the circular base, kg/cm²
D = diameter of the plunger, cm
Δ = soil deformation (the depth of indentation, cm)
ω = factor depending on the form and rigidity of the indented area (for a round plate $\omega = 1$).

The moduli of strain for roadbed soils are usually determined by means of mobile units (Fig. 102). This unit consists of two lorries *1* which act as counterweights, a thrust bar *2* fixed to the lorries, a hydraulic jack *3* provided with a pressure gauge, and a plunger *4* which is placed on the soil. The plunger has a diameter of from 60 to 30 cm.

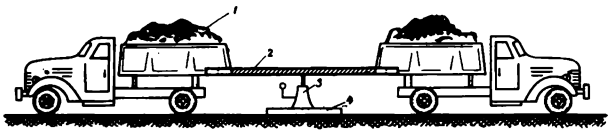


Fig. 102. Mobile unit for determining soil modulus

The loads are applied to the plunger by means of the jack, each load being maintained until the settlement stops. The settlement is then measured by means of gauges affixed to a measuring bar not connected to the test set.

The loads are applied by stages, in increments of 0.5 kg/cm², reaching an ultimate pressure of 3 to 5 kg/cm². In accordance with the results of the test, graphs are plotted giving the relation between the settlement and the specific pressure on the plunger. These graphs are then used to estimate the value of the moduli of strain with the aid of formula (165).

Since for soils there is no direct correspondence between the pressure and strain, the modulus of strain alters at various depths of indentation of the plunger. Small deformations correspond to high values of soil modulus of strain (Fig. 103).

According to formula (164), each pavement has its own critical value of flexure. Therefore, under service conditions each type of pavement will have its own individual value of bed soil modulus of strain. However, for flexible pavements the variations in the magnitude of the employed moduli of strain are not important.

An analysis of Fig. 103 shows that the greatest variation of the soil modulus of strain occurs with very small deformations which are characteristic of rigid cement concrete pavements. For the range of extensive deformations which correspond to the destructive flex-

ures for flexible pavements, variations in the magnitude of the modulus of strain are insignificant. Therefore, for the design of this type of pavement its value can be considered as constant.

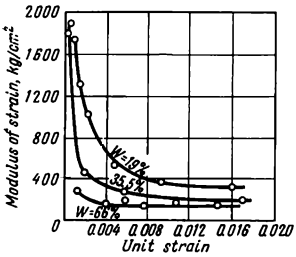


Fig. 103. Relation between soil modulus of strain and unit strain at various moisture contents

The pore-water and temperature conditions of the roadbed change throughout the year. This causes a corresponding annual pattern of change in the magnitude of the subgrade modulus of strain (Fig. 104). The lesser the possibility of pavement subgrade saturation, the greater can be the assumed design values of the modulus of strain.

The reduction of the value of the modulus of strain is especially pronounced during the wet spring season. The design values of the modulus of strain given in the instructions for the calculation of flexible pavements are related to this period. During the dry seasons of the year, and also in conditions of frozen soil, the soil modulus

Fig. 104. Variation of soil modulus of strain during year

is higher than the one given in the tables. This should be taken into account when assessing the possibility of moving heavy loads over a road during these seasons. In general, since the excess moisture

TABLE 23

Soil	Type of country, in relation to its moisture conditions	Roadbed soil modulus of strain, kg/cm², for various road zones							
		Zone II		Zone III		Zone IV		Zone V	
		Embankments	Cuttings and following ground line	Embankments	Cuttings and following ground line	Embankments	Cuttings and following ground line	Embankments	Cuttings and following ground line
Coarse sand	—	450	350	400	350	450	350	450	350
Medium sand	—	350	250	350	250	400	300	400	300
Fine sand	—	300	150	300	150	350	200	350	200
Very fine sand, sandy loam and optimum mixtures	1	150-200	120-150	170-220	150-170	200-225	170-200	240-260	220-250
	2	120-150	60-80	130-165	80-110	140-170	100-140	150-200	120-150
	3	115-140	—	120-150	—	130-160	—	140-180	—
Silty sand, fine not silty sandy loam	1	120-160	90-120	150-180	120-150	160-200	150-180	190-220	160-190
	2	80-100	—	100-125	65-80	120-140	90-120	130-160	100-120
	3	75-95	—	90-120	—	100-130	—	120-160	—
Loam, clay-loam, light and heavy clay	1	110-150	80-110	140-160	110-140	150-190	130-160	190-220	150-180
	2	75-85	—	90-115	—	110-130	75-90	130-150	90-120
	3	70-90	—	85-110	—	90-120	—	120-140	—
Silty soil, silty loam, silty sandy loam, loess	1	90-110	75-90	120-150	100-120	130-160	120-130	160-190	140-180
	2	70-80	—	85-105	—	90-120	—	125-140	85-110
	3	60-70	—	80-90	—	85-110	—	110-135	—

TABLE 24

Name of materials of pavement structural layers	Modulus of strain, kg/cm², for various zones and hydrological conditions		
	II	III	IV-V
Asphalt and tar concrete		2,600-3,000	
Sett and tesselated paving		2,500-2,800	
Aggregate mixtures treated in plants with organic binders		2,000-2,200	
Ditto, base of 3rd grade rock	1,000	1,000	1,200
Ditto, 4th grade	800	900	1,000
Ditto, 5th grade	—	—	600
Aggregate mixtures treated with organic binders:			
(a) in a plant, of 1st and 2nd grade materials		2,200	
(b) of 3rd grade materials		1,500	
(c) of 4th grade materials	1,200	1,400	1,500
Gravel and soil-aggregate mixtures treated with binders, depending on their composition		1,000-1,800	
Gravel and soil-aggregate mixtures, depending on their composition	500-900	450-900	500-1,000
Pavings and packing-course:			
(a) of 1st and 3rd grade stone		1,500-1,900	
(b) of 4th grade stone		800-1,200	
(c) of 5th grade stone		800	
Gruss, clinker, broken brick, well burnt rock		350-600	
Shell, bog ore, marl, brown coal cinder		200-400	
Soil treated with organic binders, depending on the quantity of the addition:			
(a) optimum sandy loam	—	700-800	
(b) silty soil, clay-loam, chernozem soil	—	600-700	
Soil treated with mineral binders, depending on the quantity of the addition:			
(a) optimum sandy loam	—	600-1,400	
(b) silty soil	—	400-1,200	
(c) chernozem	—	800-1,000	
(d) aggregate and gravel mixtures	—	1,600-2,000	

content of the soil in spring is not equal from year to year owing to weather variation, it would be expedient to make use of the idea of the frequency of recurrence of various bed-soil conditions when setting the design moduli of strain. The higher the road class and the greater its significance for the national economy, the higher should be the moisture content assumed for road pavement design purposes and, correspondingly, the less frequent its occurrence. Also the design frequency of the critical soil state should be co-ordinated with the service life of various pavements.

The design values of roadbed soil moduli of strain used at present, which are given in Table 23, were obtained experimentally by applying test loads to roadbed soils and carrying out check calculations in accordance with data relating to the destruction of pavements, when the character of traffic and the pavement structure were known.

Soils for which no values of the moduli of strain are given in Table 23 for roadbeds in cuttings and following the ground line, should be replaced with stable soils (see Fig. 76). The values of the moduli of strain for sands indicated in the column "Embankments" refer to fills of a height exceeding by 50 cm the possible level of capillary rise. The values for cuttings refer to sand laid in a trough.

In the road zone I, where permafrost is prevalent, the influence of local conditions (orientation of slopes, nature of vegetative cover, the depth of occurrence of permafrost) is so great that the average design moduli of strain for the soil cannot be determined. The value of these moduli should be chosen by the designer after an analysis of local conditions.

In modern towns, where a storm water network and properly-surfaced and metalled pavements are provided, the incident rainfall will be drained directly into the storm-water system. For this reason pore-water and temperature conditions are more favourable than in country areas without this provision and the soil moduli of strain values can be increased by 30%. The design moduli of strain of pavement structural layers are given in Table 24.

In all cases the higher values of the moduli of strain given in Table 24 correspond to more favourable service conditions, better hydrologic conditions, and a greater proportion of binder.

52. Calculation of Flexible Pavement Thickness

The above relationship between pavement strength and deflection, together with the known magnitude of deformation that causes complete failure permits the minimum pavement thickness to be calculated. For this end the pavement should be so designed that its deformation under the action of the maximum design load will not exceed the critical value.

The theory of flexible pavement calculation, which was developed in the years preceding the Second World War by a large group of Soviet investigators under the leadership of Prof. N. N. Ivanov, is based on the following principles.

In accordance with the method used in soil mechanics for calculating settlement, let us consider a separate column of roadbed soil having a circular base with an area equal to the area of contact of dual

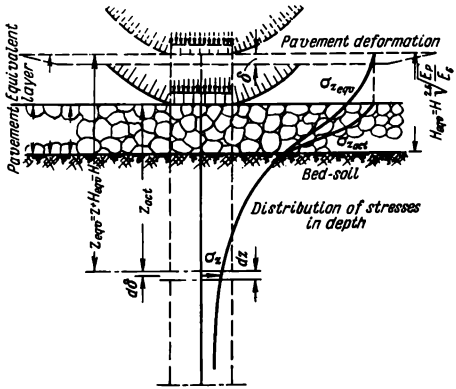


Fig. 105. To deduction of formula for determining deformation of pavement

automobile wheels with the surface (Fig. 105). The column is compressed by stresses which attenuate with depth. Let the stresses at depth z be $\sigma_z = p_0 f(z)$, where p_0 is the specific pressure of a wheel with a pneumatic tyre on the surface, and $f(z)$ is a certain function depending on the ratio of the modulus of strain of the soil E_s to that of the pavement E_p. The compression of an elementary layer of a thickness dz will be

$$d\Delta = \frac{p_0 f(z)\, dz}{E} \tag{167}$$

and the total settlement of the pavement

$$\Delta = p_0 \int_0^h \frac{f(z)\, dz}{E_p} + p_0 \int_h^\infty \frac{f(z)\, dz}{E_s} \tag{168}$$

where h is the requisite depth of the pavement, cm.

The first part of the formula characterizes the pavement compression, and the second the compression of the subgrade.

The distribution of stresses within the road pavement bed soil depends on the pavement thickness and on the ratio of the modulus of strain of the pavement to that of the bed soil. The effect of the road pavement on the distribution of stresses in the bed soil can be determined by employing the theory of "equivalent layer", which was suggested by Prof. G. I. Pokrovsky. The idea of this theory can be explained by the following simplified example.

Imagine that resting on the soil are two long beams of different materials but having equal widths. It is evident that the distribution of stresses in the soil under the beams will be identical only in the case when their deflections are equal under the action of equal loads. This necessitates that the rigidities of the beams be equal, i.e.,

$$E_1 I_1 = E_2 I_2$$

where E = modulus of strain, kg/cm²
$I = bh^3/12$ is the moment of inertia, cm⁴
b = width of the beam, cm
h = depth of the beam, cm.

Substituting the values for I, after transformation we obtain

$$h_1 = h_2 \sqrt[3]{\frac{E_2}{E_1}} \tag{169}$$

This relation can be also applied to the distribution of stress in the road pavement bed soil. For this purpose it is necessary to replace the surfacing layer by an imaginary *equivalent layer of soil* of such a depth, that the stresses at the level of the subgrade surface will remain the same (Fig. 106).

Since the strain of the soil and of the pavement materials is not proportional to the pressure, the formula of the equivalent layer for calculating the stresses in the road pavement base was replaced, in accordance with experimental data, by an empirical expression

$$h_s = h_p \sqrt[2.5]{\frac{E_p}{E_s}} \tag{170}$$

According to experimental data, this formula is valid for the following ratio:

$$\frac{E_p}{E_s} \leqslant 35$$

The processing of experimental data obtained by a number of investigators who measured the stresses in flexible pavement bases shows that the distribution of stresses in depth corresponds sufficient-

ly well to the empirical formula

$$\sigma_z = \frac{p_0}{1+\left(\frac{z_{eqv}}{D}\right)^2} \tag{171}$$

where p_0 = specific pressure of an automobile wheel on the surfacing, kg/cm²

z = equivalent depth of the point under consideration when the road pavement is replaced by an equivalent layer of soil, cm

D = diameter of circle equivalent to the area of an imprint of dual vehicle wheels, cm.

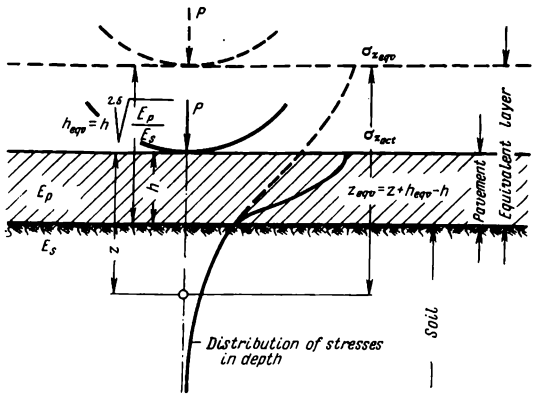


Fig. 106. Equivalent layer of soil

Upon replacing in the equation (168) the expression $f(z)$ by the function σ_z from the equation (171), we obtain the initial expression for calculating the settlement

$$\Delta = \frac{p_0}{E_p}\int_0^h \frac{dz}{1+\frac{z_{eqv}^2}{D^2}\sqrt[2.5]{\left(\frac{E_p}{E_s}\right)^2}} + \frac{p_0}{E_s}\int_h^\infty \frac{dz}{1+\frac{1}{D^2}\left[z+h\sqrt[2.5]{\left(\frac{E_p}{E_s}\right)^2}-1\right]} \tag{172}$$

After integration and substitution of limits the following expression for determining the magnitude of pavement deflection is obtained:

$$\Delta = \frac{pD}{E_s}\left[\frac{\pi}{2} - \left(1 - \frac{1}{\sqrt[2.5]{\left(\frac{E_p}{E_s}\right)^{3.5}}}\right)\arctan\frac{h}{D}\sqrt[2.5]{\frac{E_p}{E_s}}\right] \quad (173)$$

Knowing the magnitude of the maximum deflection, and also the characteristics of the pavement and of the soil, this formula can serve for determining the thickness of a single-layer pavement laid on a homogeneous soil (the so-called double-layer system) for a given load.

It is not practicable to use equation (173) for finding the thickness of the layers of a multilayer pavement without the introduction of additional assumptions. For a homogeneous material the extent of deflection can be determined by assuming the thickness of the upper layer $h = 0$ in formula (173), then

$$\Delta = \frac{\pi p_0 D}{2E} \quad (174)$$

Making use of formulas (173) and (174), one can now replace the modulus of strain for a pavement consisting of several layers of various thickness and of various quality by the modulus of strain for a hypothetical homogeneous material which would be subject to the same degree of deflection. This modulus is called the *equivalent modulus of strain.*

For determining the equivalent modulus of strain of a single-layer surfacing laid over a homogeneous soil, formulas (173) and (174) are equated, whence, after transformation, we obtain

$$E_{eqv} = \frac{E_s}{1 - \frac{2}{\pi}\left(1 - \frac{1}{\sqrt[2.5]{\left(\frac{E_p}{E_s}\right)^{3.5}}}\right)\arctan\frac{h}{D}\sqrt[2.5]{\frac{E_p}{E_s}}} \quad (175)$$

For practical application of this formula a graph is provided (Fig. 107).

The necessity for determining the equivalent modulus of strain of a multilayer pavement, making use of the known thicknesses of the separate layers, arises when it is required to estimate the strength of existing pavements at the time of road reconstruction. For the design of a new road pavement structure it is necessary to proceed in the reverse order—knowing the maximum permissible degree of flexure and the relevant equivalent modulus of strain, the thickness

of structural layers to provide the necessary modulus is found. This problem can be solved by means of the graph.

The concept of the pavement equivalent modulus permits the evaluation of the strength of pavements by means ot a single index.

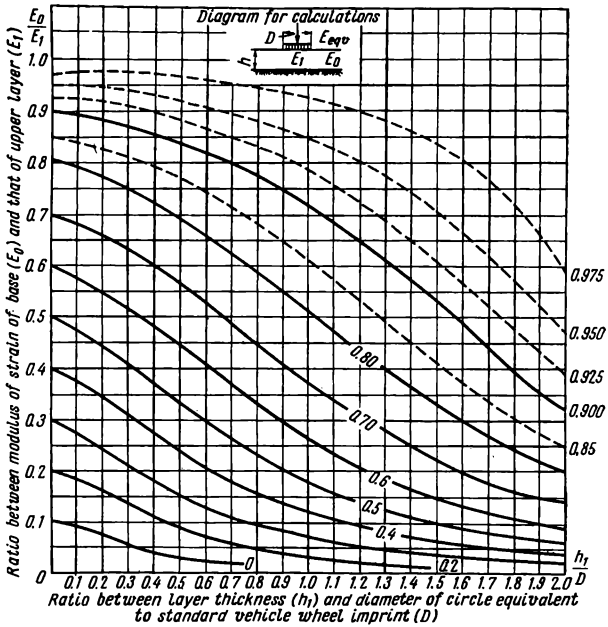


Fig. 107. Graph for computation of road pavement thickness (the figures along the curve give the E_{equ}/E_1 ratio for which the curve is plotted)

In particular, to provide for the required pavement margins of safety on roads of various types, and to allow for future growth of traffic intensity, it is considered that the equivalent moduli of strain for pavements on roads of various classes should not be less than those given in Table 25.

The degree of permissible flexible pavement deformation considered above is based on a single application of an external load. In

TABLE 25

Road class	Equivalent moduli of strain, kg/sq cm, in relation to type of pavement		
	High-quality heavy-duty	High-quality light-duty	Intermediate
I	700	650	—
II	600	600	—
III	560	500	—
IV-V	—	380	300

reality, a road carries a daily flow of a very large number of vehicles. Their repetitive action in periods of excessive moistening of the subgrade gives rise to a cumulative plastic deformation of the road pavement, causing first the disturbance of its inner structure and finally its failure. Road investigations show that pavements which had an appreciable strength as calculated for a single load application, were destroyed by recurrent application of lesser loads. This phenomenon can be explained by the fact that the recurrent application of loads gradually lowered the resistance of the pavement by causing the accumulation of permanent deformations and a reduction in the value of the equivalent modulus. The greater the number of load applications, the more intensive is the reduction of pavement strength.

On the basis of experimental data, an empirical relation has been established

$$E_{eqv_N} = E_{eqv.static} K = \frac{E_{eqv.static}}{0.5 + 0.65 \log \gamma N} \qquad (176)$$

where E_{eqv_N} = pavement equivalent modulus of strain due to action of N vehicles per day on a single lane

$E_{eqv.\,static}$ = equivalent modulus of a newly built pavement

K = factor taking account of the traffic intensity

γ = factor taking into account the influence of the carriageway width on the frequency of load action: for roads with two-lane traffic $\gamma = 1$; for roads with single-lane traffic, when the vehicles moving in both directions pass over a single tread path, $\gamma = 2$; with a four-lane carriageway $\gamma = 0.45$.

The required magnitude of the equivalent modulus of strain for a pavement intended for carrying daily N vehicles per lane, accord-

ing to equation (174), is equal to

$$E_{eqv_N} = \frac{\pi p D}{2\Delta_{per}} \tag{177}$$

whence, according to equation (176), during construction the equivalent modulus of strain has to be

$$E_{eqv.static} = \frac{\pi p D\,(0.5 + 0.65 \log \gamma N)}{2\Delta_{per}}\,\mu \tag{178}$$

The factor μ is called the safety factor for nonuniform conditions of pavement performance; for pavements with heavy-duty surfacings $\mu = 1.2$; for pavements with high-quality light-duty surfacings $\mu = 1.1$; for pavements with intermediate surfacings that can be easily repaired, $\mu = 1.0$.

For two different types of vehicles, an equal value of the required equivalent modulus corresponds to a different design traffic intensity.

For vehicles of the first type (standard)

$$E_{eqv.static} = \frac{\pi p_1 D_1}{2\Delta'_{per}}\,(0.5 + 0.65 \log \gamma N_1) \tag{179}$$

For vehicles of the second type

$$E_{eqv.static} = \frac{\pi p_2 D_2}{2\Delta''_{per}}\,(0.5 + 0.65 \log \gamma N_2) \tag{180}$$

Since vehicles of many classes use the roads, the traffic intensity should be reduced to that of an equivalent number of standard vehicles.

By equating the expressions (179) and (180) it is found that the traffic intensity of the second type of vehicles can be reduced to the equivalent traffic intensity of standard vehicles

$$\log \gamma N_1 = \frac{p_2 D_2 \Delta'_{per}}{p_1 D_1 \Delta''_{per}}\,(\log \gamma N_2 + 0.77) - 0.77 \tag{181}$$

The total intensity, expressed in standard vehicles, will be: $N = N'_1 + N''_1 + \ldots$,
where N'_1, N''_1 are the equivalent traffic intensities.

To change from one type of vehicles to another, one can use the graph shown in Fig. 108.

The thicknesses of individual layers in a flexible pavement are calculated as follows:

1. Depending on the availability of local road-building materials, and also on the class and purpose of the road, a road pavement structure is selected and the initial thickness of separate layers is estimated. These layers are selected on considerations other than those

concerning the pavement strength (material cost, prevention of frost heaves, etc.).

It is desirable that for all versions of multilayer pavements all the layers, except the sand base or one of the layers which determines

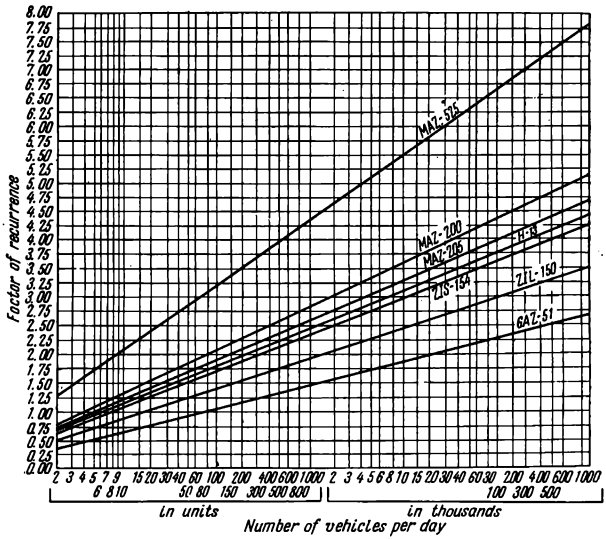


Fig. 108. Graph for converting mixed traffic flow to intensity of standard vehicle traffic

the pavement strength—the bearing course—should be selected according to structural considerations.

2. The formula (178) is then used for determining the pavement equivalent modulus of strain.

3. By means of the graph (see Fig. 107) the equivalent moduli of the structural layers situated below the pavement are determined.

Imagine, for example, a three-layer structure (surfacing, base, sub-base) in which the thickness of the base h_2 is unknown. The thickness of the surfacing h_1 is chosen in accordance with structural considerations.

First, the required equivalent modulus is determined for the structure consisting of layers h_2 and h_1 and the sub-base. For this purpose, the ratios E'_{eqv}/E_3 and h_3/D

are assumed and, making use of the graph, the ratio E''_{eqv}/E_3 is found from which the value of E''_{eqv} can be determined.

The equivalent modulus of the layers situated below the unknown layer is found as follows: using the ratios E_0/E_4 and h_4/D, locate on the graph the ratio E'_{eqv}/E_1, from which the value of E'_{eqv} may be computed.

Finally, by means of the values E''_{eqv}/E_2 and E'_{eqv}/E_2 locate on the graph the ratio h_2/D, which gives the unknown quantity h_2. If the resultant thickness of the layer h_2 is found to be inadmissible because of structural considerations, the thicknesses of the other layers will have to be amended accordingly, and the calculation repeated.

53. Determination of Rigid Pavement Thickness

Rigid road pavements include cement concrete* and reinforced concrete surfacings and bases, which are capable of sustaining the tensile stresses occurring during flexure.

By distributing the vehicle wheel pressure over a large area, the concrete surfacings transmit relatively low pressures to the bed soil. However, the influence of the bed soil resistance to external loads on the performance of concrete pavement is not less than that of the strength of the concrete structure itself.

For a long time it was considered that the strength of the subgrade had little influence on the resistance of concrete surfacings to loads. However, it was established experimentally that such assumptions were entirely false.

The strength of concrete surfacings depends to a very large degree on the uniformity of subgrade resistance over the whole base area of the slab.

A lack of uniformity in the compaction of the sand layer or of the roadbed leads to a situation when, owing to the non-uniform settlement of the subgrade material, part of the slab will tend to be without direct support from the subgrade, causing the flexural stresses to increase sharply in comparison with the design ones.

For designing rigid road surfacings and bases subject to the action of external loads, the theory of beams and slabs on an elastic foundation, which has been developed in structural mechanics, is employed. A significant contribution to the development of this theory was made by the Soviet scientists—academician A. N. Krylov, Prof. M. I. Gorbunov-Posadov, B.N. Zhemochkin and O.Y. Shekhter.

When laying the concrete mixture within the specially prepared trough formed in the roadbed, the mixture is divided by spacers in order to obtain rectangular slabs of a width equal to the full width of the carriageway, or to the operative width of the mechanical

* Here and in the following, cement concrete surfacings and bases will be called "concrete".

spreader. When laying surfacings of precast slabs fabricated off-site, these are made of smaller dimensions, in accordance with the capacity of the vehicles used to deliver them.

The thicknesses of concrete slabs are determined in accordance with the loads to be sustained. The selected dimensions, particularly the length in the case of rectangular slabs, should be checked for temperature stresses. The thickness of the concrete surfacings is calculated in the same way as for flexible pavements, viz., in accordance with the load of a wheel of a standard (H-13) vehicle ($p = 4{,}550$ kg, $D = 34$ cm), with the introduction of the dynamic factor 1.25 to allow for impact loads when passing over surface irregularities and for vehicle vibration during motion.

Calculation of stresses in slabs due to external loading. For this calculation it is necessary to determine the most unfavourable position of a vehicle wheel running over the surface of the slab. From the design point of view three locations are possible for the point of application of wheel pressure to a rectangular slab, i.e., *at the centre of the slab, at a corner, or at an edge.*

In a single concrete slab the maximum stresses occur when the load is acting on the edge or on the corner. However, this case is the most complicated one for theoretical analysis, and to date there are no solutions for it based on the characteristics of the subgrade. For this reason the strength of concrete surfacings is often calculated for the case of load application in the central part of the slab, and the value of the induced moments arising when the load is applied at the corner or at the edge, is estimated by the introduction of suitable correction factors.

In the case of load application in the central part of a fairly large slab, when the deflection wave does not reach the edges, it is possible to apply theoretical solutions obtained for an infinite slab on an elastic foundation, the most accurate of which is the solution proposed by O.Y. Shekhter. When a concrete slab is subject to the action of a concentrated load or a load uniformly distributed over a circular area, radial and peripheral moments are induced. The magnitude of these moments depends on the size of the load and on the rigidity of the slab. The latter is characterized by the following parameter:

$$a = \frac{1}{H}\sqrt[3]{\frac{6E_s(1-\mu^2)}{E_c(1-\mu_s^2)}} \approx \frac{1}{H}\sqrt[3]{\frac{6E_s}{E_c}} \qquad (182)$$

where H = thickness of the concrete slab, cm

E_c = modulus of elasticity of the concrete surfacing, kg/cm²

E_s = modulus of strain of the bed soil (if the base consists of several layers with heterogeneous properties, then E_s is the equivalent modulus of strain of the bed soil), kg/cm²

μ_s and μ = Poisson's ratios for soil and pavement, respectively.

The cubic root of the ratio of the members containing μ is approximately equal to unity.

In accordance with the above and having regard to the difficulties in evaluating the reaction of the subgrade, concrete pavement thicknesses are calculated assuming a soil modulus of strain 2.5 to 4 times greater than would be used for flexible pavements.

The flexural moments acting on a strip of unit width are:

(a) due to a load uniformly distributed over a circle of radius R

$$M_{rad} = \frac{CP(1+\mu)}{2\pi aR} \tag{183}$$

(b) due to a concentrated force

$$M_{rad} = (A + \mu B)P \tag{184}$$

$$M_{periph} = (B + \mu A)P$$

where P = concentrated load or the resultant of a uniformly distributed pressure, kg

μ = Poisson's ratio for concrete

C = factor depending on the product aR

A and B = parameters depending on the product ar

r = distance from the point of application of the concentrated load to the point at which the stress is determined, cm.

In formula (183) $P = p\pi R^2$, where p is the intensity of uniformly distributed loading.

The values of parameters A, B and C are given in Table 26.

TABLE 26

ar or aR	Values of parameters			ar or aR	Values of parameters		
	A	B	C		A	B	C
0							
0.05	—	—	0.091	1.4	0.038	—0.017	—
0.1	0.232	0.153	0.147	1.6	0.031	—0.019	0.309
0.2	0.178	0.099	0.220	1.8	0.025	—0.019	—
0.3	0.147	0.068	0.275	2.0	0.021	—0.020	0.263
0.4	0.124	0.047	0.313	2.2	0.017	—0.019	—
0.6	0.093	0.021	0.352	2.4	0.014	—0.018	—
0.8	0.075	0.004	0.367	2.6	0.012	—0.017	—
1.0	0.058	—0.006	0.364	2.8	0.010	—0.016	—
1.2	0.047	—0.013	0.353	3.0	0.008	—0.014	—

When calculating the moments due to wheel loading one uses the formula for loading distributed over a circular area.

When designing road pavements and bases one often has to calculate the effect of heavy loads such as rollers, multiwheel trailers or track-laying plant, in which case the formula for a circular plunger cannot be applied. Here designers must resort to the method of integrating the stresses induced by a train of concentrated loads. The pressure applied to the contact area of the load with the surfacing is then replaced by a number of concentrated forces applied to the

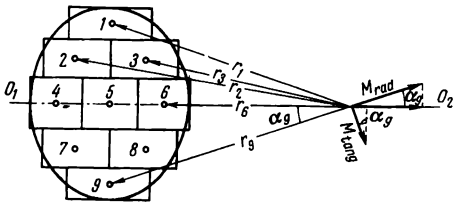


Fig. 109. Replacement of loading uniformly distributed over a number of areas with concentrated forces

independent centroids of individual areas (Fig. 109). The flexural moment at the point at which one has to determine the stresses acting on a strip of surfacing of unit width, is calculated as the geometrical sum of the flexural radial and peripheral moments of all the concentrated forces.

When determining the components of the moments of the various forces account must be taken of the projections of not only the flexural moments proper, but also of the pavement strips on which they act (Fig. 110). Imagine a strip of surfacing AA_1 of unit width, on which a flexural moment M is acting. The projection of this moment on the direction BB_1, which is at an angle α to AA_1, is $M\cos\alpha$, and the projected width of the strip to which it is applied increases to $1/\cos\alpha$. Therefore, the moment acting on a strip of unit width in the direction AA_1 is

$$M\cos\alpha : \frac{1}{\cos\alpha} = M\cos^2\alpha$$

Therefore the formula for determining the total moment is as follows:

$$M_{fl} = M_{rad}\cos^2\alpha + M_{periph}\sin^2\alpha \qquad (185)$$

where α is the angle between the centre line about which the moments are applied, and a line joining the point of application of the force with the point at which the stresses are being investigated.

If the angle α is below 20°, one may assume $\cos^2 \alpha = 1$, and $\sin^2 \alpha = 0$ and sum—without any appreciable error—the radial flexural moments, neglecting the value of their projection.

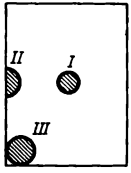


Fig. 110. Cases of load application used in analysis:
I—central loading; *II* and *III*—edge loading

The stresses in concrete due to a flexural moment on a strip of a unit width are determined by the usual structural mechanics formula

$$\sigma = \frac{6\Sigma M}{H^2} < \sigma_{max} \qquad (186)$$

The thickness of the slab should be selected to ensure that the stresses developed will not exceed the maximum permissible values. The allowable stresses induced by live loads are taken at 0.5-0.6 of the ultimate concrete flexural strength.

If the concrete mixture is laid on a sand bedding which deforms during the process of construction, and into which cement grout may partially penetrate, the computed thickness of the slab is increased by 1 cm.

The application of a load at a corner of a slab or at its edge (Fig. 111) will induce greater stresses than application of the load at the centre.

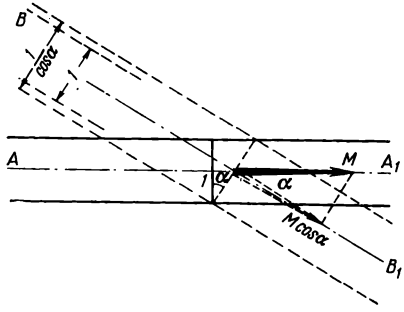


Fig. 111. To determining total moment when several loads act on a slab

The magnitude of these moments can be determined by means of a method proposed by I.A. Mednikov.

Westerhard's formulas as defined more precisely by Mednikov are as follows:

(a) For a wheel on the edge of a slab

$$\sigma_e = \alpha_2 \frac{p}{h^2}$$

(b) For a wheel on the corner of a slab

$$\sigma_c = \alpha_3 \frac{p}{h^2}$$

The values of factors α_2 and α_3, which depend on the ratios h/R and E/E_0, are given in Table 27.

TABLE 27

E/E_0	Values of factors α for the following ratios of h/R				
	2	1.6	1.2	0.8	0.5
	Values of factors α_2				
2,000	2.44	2.49	2.21	1.82	1.39
1,500	2.62	2.42	2.14	1.71	1.32
1,000	2.51	2.29	2.00	1.60	1.19
500	2.26	2.03	1.75	1.39	0.98
200	1.97	1.73	1.49	1.13	0.69
100	1.73	1.54	1.29	0.92	0.50
	Values of factors α_3				
2,000	2.37	2.25	2.09	1.80	1.49
1,500	2.31	2.40	2.04	1.73	1.42
1,000	2.26	2.13	1.95	1.66	1.34
500	2.11	1.97	1.75	1.49	1.14
200	1.92	1.76	1.57	1.28	0.87
100	1.76	1.62	1.41	1.08	0.69

In practice calculations are made for the load application in the centre, and the slabs are reinforced at the corners and edges, together with load transference by dowels. In the longitudinal joint of the slab two steel dowels 12 to 16 mm in diameter are located 5 cm above the underside of the slab and at a distance of 10 and 30 cm from the end. The dowels are continued through the joints. The corners of the slabs are often reinforced by placing bent reinforcement bars along the edges and at an angle to them of 30°.

When analyzing small slabs of precast concrete surfacings, which are usually of hexagonal or rectangular form, the calculation is based on an equivalent circular slab of the same area.

According to the classification of the Soviet scientist Gorbunov-Posadov, all slabs can be divided into three categories, depending on the value characterizing their stiffness

$$S = \frac{3E_s R^3 (1-\mu^2)}{E_c h^3 (1-\mu_s^2)} \tag{187}$$

where E_s = soil modulus of strain, kg/cm²
E_c = concrete modulus of elasticity, kg/cm²
R = radius of circular slab, cm
h = slab thickness, cm.

When $S > 10$ the slabs are assumed to be infinite and analyzed by the method of O.Y. Shekhter. Their deflection wave is confined to the central part of the slab. The size of the slab and the degree of fixation of the edges have no influence on the magnitude of stresses.

When $0.5 \leqslant S \leqslant 10$ the slabs are classified as *finitely rigid.*

When $S < 0.5$ the slab behaves under load as a uniform rigid slab subject to settlement, i.e., the settlement of all the surface points of which is identical. Such slabs are called *infinitely rigid.*

The moments at the centre of finitely rigid and infinitely rigid slabs induced by the load distributed over a circular area at the centre of a circular slab, are determined according to the formula

$$M_r = M_t = (\overline{M}_A + \overline{M}_B) P \tag{188}$$

where $P = \pi p r_0^2$ is the resultant of a load uniformly distributed over a circle of a radius r_0

$\overline{M}_A$ and $\overline{M}_B$ = parameters which depend on the slab stiffness factor $\overline{S}$ and the ratio r_0/R, the values of which are given below.

Values of parameter $\overline{M}_A$

S	< 0.05	0.5	1	2	3	5	10
$\overline{M}_A$	0	−0.052	−0.056	−0.066	−0.074	−0.086	−0.108

Values of parameter $\overline{M}_B$

$\alpha = r_0/R$	0.005	0.01	0.02	0.03	0.04	0.05	0.075	0.10	0.15
$\overline{M}_B$ for $S < 0.5$	0.532	0.468	0.403	0.366	0.339	0.318	0.280	0.251	0.215
$\overline{M}_B$ for $S \geqslant 0.5$	0.571	0.507	0.443	0.405	0.378	0.358	0.320	0.293	0.255

The stresses induced in the centre of the slab by a single wheel are calculated by means of the usual formula

$$\sigma = \frac{6M}{H^2} \tag{189}$$

The choice of the allowable stresses is related to the time elapsing between construction and opening of the carriageway to traffic, including constructional traffic. The strength of concrete increases with time. It is therefore necessary that the values of the concrete strength and of its modulus of elasticity assumed in calculations should correspond to the actual time of appearance of the design load on the road. If necessary, the designer can make use of special methods available for accelerating the hardening and curing of concrete, e.g., the use of special cements, vacuum treatment, the introduction of additives to accelerate hardening, or heating of the concrete.

By making use of the relationship between concrete strength and ageing time as determined by laboratory tests, it is also possible to employ different pavement thicknesses on various sections depending on the time which will elapse from the moment of laying the surfacing on the given section to its opening to traffic.

The rate of growth of concrete strength is given by the following empirical formula:

$$R_n = R_{28} \frac{\log (n+1)}{\log 28} \tag{190}$$

where R_n and R_{28} is the ultimate compressive strength after n and 28 days, respectively.

In urban areas the concrete bases are usually surfaced with a layer of asphalt concrete, which reduces the stresses in the concrete in accordance with the following considerations:

1. The layer of asphalt concrete distributes the pressure on the concrete base over an area somewhat larger than the design tyre imprint. As an approximation it can be assumed that the pressure is distributed through the asphalt concrete layer of thickness H at an angle of 45°. Then the apparent diameter of the circle transmitting the pressure is

$$D_{app} \approx D + 2H \tag{191}$$

2. The asphalt concrete adhering to the concrete base takes up some of the stresses formerly induced in the concrete by shear stresses developed on the contact face. The flexural moment is distributed between the surfacing and the base in proportion to their rigidity.

Calculation of temperature stresses in rigid pavements. Temperature stresses arise in rigid pavements as a result of friction between the slab and the bed soil due to expansion or contraction induced by temperature change, and also because with nonuniform heating throughout their thickness the slabs cannot warp owing to their mutual wedging and the counteraction of their weight.

Thus, the occurrence of temperature stresses is connected with externally induced resistance to freedom of longitudinal strain with a temperature change. To reduce the temperature stresses the size

of the slab may be reduced to that at which these stresses are insignificant.

When laying the concrete mix, as a result of thorough compaction, individual chips are pressed into the sub-base, thus forming an uneven lower surface of the pavement. It can be assumed that with the temperature shrinkage or expansion of the slab its centroid remains in the same place and its edges move freely. Thus, the deformation

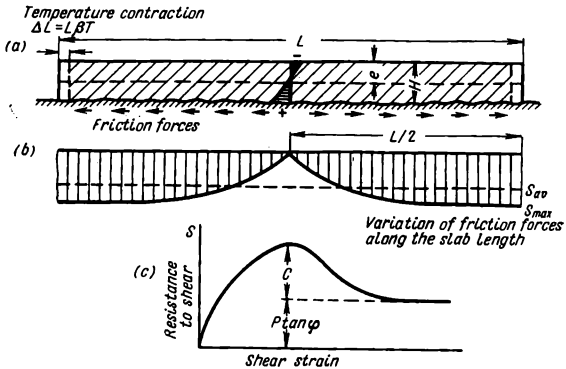


Fig. 112. To determining slab length:
a—appearance of friction forces along slab base; b—variation of friction forces along slab base; c—resistance to shear versus strain

steadily increases from the centre of the slab to its edges. In order to move, the slab has to overcome the soil shear resistance along the contact surface between the slab and the soil. One should also keep in mind that the resistance of soil depends on the amount of shear strain, the resistance growing along a parabolic curve within certain limits (Fig. 112).

For practical purposes it can be accepted that at the slab edges the soil resistance to shear attains its maximum value:

$$S_{max} = p \tan \varphi + C \tag{192}$$

where p = slab pressure on the soil, equal to $H\gamma$
γ = unit weight of the slab, kg/cm³
H = slab thickness, cm
φ = angle of internal friction
C = soil cohesion, kg/cm².

Since the maximum stresses in the concrete occur during periods of maximum slab heating or cooling, the values of φ and C refer to dense dry or frozen soil. Then, according to the parabolic characteristics, the mean value of resistance along the slab-to-soil contact surface will be the following:

$$S_{av} \approx 0.7 S_{max} = 0.7\,(H\gamma \tan\varphi + C) \tag{193}$$

The total soil resistance to slab displacement is therefore

$$S = S_{av} BL = 0.7BL\,(H\gamma \tan\varphi + C) \tag{194}$$

Since this stress is applied to the lower surface of the slab, it results in eccentric compressive stresses appearing in the slab cross-sections, i.e.,

$$\sigma = \frac{S}{BH}\left(1 + \frac{6e}{H}\right) \tag{195}$$

The maximum fibre distance from the neutral axis of the slab is $e \approx H/2$; hence the maximum value of the tensile stress becomes

$$\sigma = \frac{2S}{BH} \tag{196}$$

whence the maximum length of slab L becomes

$$L = \frac{\sigma H}{1.4\,(H\gamma \tan\varphi + C)} \tag{197}$$

The values of the shear resistance have been determined by a number of investigators. The values of C and $\tan\varphi$ for various types of bases are as follows:

	C	$f = \tan\varphi$
Loamy soil	0.7	1.0
Sand bedding	0.3	0.7
Pergamin course	0.5	0.9
Blast furnace slag	0.9	0.8
Rubble	0.2	1.2

A temperature gradient appears over the thickness of the slab when it is heated by sun rays or cooled during the night temperature fall. The difference between the temperatures of the upper and lower slab surfaces may be as much as 20 to 30°C. The heated surface expands, as a result of which the slab warps, tending to produce a curved surface. When the upper surface is heated the slab tends to become convex, and when it is cooled its surface is concave. However, free slab warping is prevented by the slab weight and by the restraining action of the edges, since during flexure the joints close, while restraint is also exercised by the dowels. The suppression of warping gives rise to additional temperature stresses in the slabs.

According to Westerhard the temperature stresses which arise in concrete pavement slabs as a result of the prevention of warping are: at the slab edge

$$\sigma_t = \frac{C_x E_c e_t \Delta t}{2} \approx C_x \Delta t \tag{198}$$

in the middle of the slab

$$\left.\begin{aligned} \sigma_{t\,max} &= \frac{E_c e_t + \Delta t}{2(1-\mu^2)}(C_x + \mu C_y) \approx \Delta t\,(C_x + \mu C_y) \\ \sigma_{t\,min} &= \frac{E_c e_t + \Delta t}{2(1-\mu^2)}(C_y + \mu C_x) \end{aligned}\right\} \tag{199}$$

In these formulas ε_t = linear stress distribution factor for the concrete

E_c and μ = concrete elastic modulus and Poisson's ratio, respectively

C_x and C_y = parameters depending on the horizontal dimensions of the slab and its rigidity. The values of C_x and C_y are obtained from a diagram (Fig. 113)

Δt = design temperature drop.

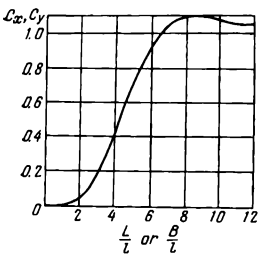


Fig. 113. Values of coefficients C_x and C_y

According to Westerhard the pavement rigidity characteristic (radius of relative rigidity) l is determined by means of the formula

$$l = \sqrt[4]{\frac{E_c h^3}{12(1-\mu^2)k}} \approx 0.6h\sqrt[3]{\frac{E_c}{E_s}} \tag{200}$$

where k is the modulus of subgrade reaction.

The calculations according to Westerhard's formula demonstrate that temperature stresses must be taken into account when the slab dimensions are greater than 4×4 m. Slabs having one side longer than 10 metres can be destroyed by the action of temperature stresses even when there is no externally applied load. In a correctly designed slab the sum of stresses due to external loading and to temperature should not exceed 0.8 to 0.9 of the ultimate concrete flexural strength.

PART IV

Route Location

CHAPTER 9

CHOICE OF ROUTE LOCATION

54. Effect of Traffic Intensity and Volume on Route Location

In modern highway design practice two essentially distinct cases of route location are possible:

1. The design of expressways or freeways, i.e., high-speed roads with limited access, when the general route and the main intermediary towns and cities on it are established with a view to national administrative and cultural considerations. These highways are used predominantly for fast through traffic. The catering to local traffic requirements, arising out of the proximity of the highway to minor industrial centres and inhabited localities, receives secondary consideration in locating the route.

2. The design of approach roads leading from industrial enterprises, agricultural communities, mines and other terminal points centering on existing roads, railway stations and ports, and also of a road network connecting a series of commodity originating and consuming centres in an industrial or agricultural area. The location of such roads is subordinated to the requirements of local traffic flows.

The route of an approach road or of a road network to handle local traffic has to be so planned in relation to the served inhabited localities and other traffic originating points, as to provide the most direct routes for traffic and thus keep the traffic ton-kilometres down to a minimum, and to provide convenient passenger services.

The elementary principle of making the route follow a bee line is relevant only in the case of connecting two major inhabited localities. When there are a great number of intermediary points, a zig-zag line or network of routes appears, which cannot be located without an assessment of potential traffic volumes and directions of flow.

The initial solution of this problem consists in finding for every case such a network of bee lines connecting the various points, which would ensure the lowest possible ton-kilometres for the given amount of traffic. This network is planned diagrammatically without consideration of the topography or situation of the country. The alignment of the bee lines serves only as a general orientation for the location survey.

As a criterion for comparing the alternative network locations the *minimum ton-kilometres* or *transportation work*, or the *minimum journey time required for the transportation of passengers and goods* can

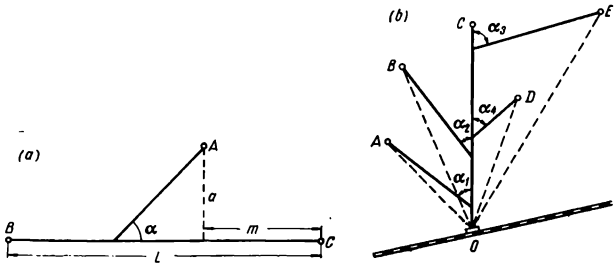


Fig. 114. To technical and economic justification of route location: *a*—determination of approach road junction with highway; *b*—determination of approach road direction

be accepted. The first criterion should be preferred when goods traffic prevails, and the second one in the case of predominantly passenger traffic.

Let us consider several characteristic problems associated with the location of local road networks.

1. *Location of an approach road junction* to a highway (Fig. 114*a*). Between the points A and B—also between B and A—a certain total volume of goods Q_1 is transported annually. Similarly, between C and A the total volume of goods is Q_2 gross tons. Let the resistance to motion be f_1 on the approach road, and f_2 on the highway. The total work is thus

$$F = \frac{(Q_1+Q_2)\,a f_1}{\sin\alpha} + Q_2 f_2 (L - m - a\cot\alpha) + (m + a\tan\alpha)\,Q_1 f_2$$

The magnitude of the junction angle α, corresponding to the minimum work, can be found by equating the first derivative $\frac{dF}{d\alpha}$ to zero,

whence

$$\cos \alpha = \frac{f_2 (Q_1 - Q_2)}{f_1 (Q_1 + Q_2)} \tag{201}$$

2. *Determination of the direction of an approach road* (Fig. 114*b*). If there are several inhabited localities and industrial enterprises served by one major centre, viz., a railway station, a port, or a large manufacturing plant, it will not be expedient to build an independent

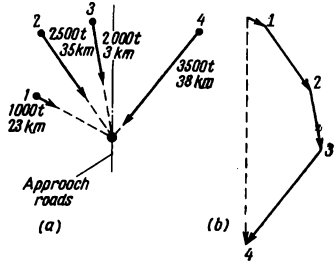


Fig. 115. Determination of approach road direction:

a—direction of traffic. The figures next to the arrows indicate traffic volumes and lengths of haul; *b*—plotting of traffic polygon

road from the centre to each of these points. The most economical solution might be to lay an arterial road *CO* with branches to the various points.

If the volume of transportation work between each of the terminal points and the centre is represented by a vector, orientated parallel to the line connecting those two points, then the direction of the arterial approach road can be obtained graphically by plotting a polygon. When constructing a force polygon in structural mechanics, vectors are plotted successively, each one from the end of the previous one, parallel to the lines of action of the forces, and with a length corresponding in the selected scale to the magnitudes of the forces. The closing line of such a diagram will give the magnitude and direction of the resultant force. In a similar manner, by replacing the magnitude of the force vector by the amount of work and the direction of the vectors by the relevant traffic lines, a traffic polygon may be drawn (Fig. 115). The closing line of such a traffic polygon—the "resultant"—will indicate the most favourable direction for the approach road. One terminal of the approach road will be the centre on which the traffic polygon was located. When planning a network intended to serve as an outlet for a product uniformly distributed

over an area (timber, agricultural produce, etc.), the centroids of individual tracts are assumed as control points. The traffic volume is determined by means of economic surveys.

The above methods are simple but limited in application, and their importance for route location should not be overrated. They give only an indication of the rational route direction, since they do not take fully into account the variations in traffic intensity with time, the types of road surfacing, the appearance of new terminal points, etc. The building of every new road contributes to the development of the economy of adjacent regions and stimulates the appearance of new traffic flows—generated traffic which may not have been foreseen and allowed for during the period of road planning. Departures of the network from the theoretical bee lines become necessary owing to local topographic conditions, but the general principles which underlie the described methods should always be considered when planning a road network.

55. Influence of Natural Conditions on Route Location

The choice of the route location determines the disposition of all the road structures. When finalizing the road location on the ground, everything possible should be done to minimize the influence of adverse local natural conditions on the construction and subsequent operation of the road. When establishing the influence on the road of topographic, geological, hydrologic and meteorological conditions one should determine the probable effect of the natural processes and the changes that are likely to occur after the construction of the highway. It is also necessary to visualize and try to take into account all the subsequent changes that may take place, e.g., the construction of reservoirs, swamp drainage, irrigation works, afforestation, etc.

The complexity and insufficient knowledge of the methods used to ensure the stability of the roadbed on stretches situated in unfavourable geological conditions are the main reason why in the majority of cases it is preferable to bypass an unfavourable region, unless this leads to a substantial lengthening of the route.

Modern mechanized methods employed for earthworks permit the building of a stable roadbed in various soil conditions, therefore soils are often analyzed mainly from the aspect of liability of frost heave.

With regard to meteorological conditions, special importance must be attached to the direction of the prevailing winds, on which depends the likelihood of road blocking by snow drifts in winter or by sand drifts in desert regions.

The natural pattern of rivers and streams may dictate the choice of crossing points and the required dimensions of bridges, and in some cases determine the practicability of locating the highway along a river valley. Appreciable water discharges at river crossings may require the construction of extensive and costly structures and justify the diversion of the route at a higher level and nearer to the watershed.

Road performance is greatly influenced by its direction in relation to the cardinal points. The amount of solar heat absorbed by slopes differs greatly according to aspect. Southern slopes will get rid of snow before the northern ones and dry out quicker than the latter—other things being equal. According to calculations by the Soviet scientist B.N. Vedenisov, the southern slope of a cutting, with the sun situated at 30° above the horizon, absorbs 14 times more solar heat than the one with a northern aspect. Earth roads which are located on southern slopes of land have a shorter slush season. To improve the roadbed drainage in wooded marshland, it is recommended to site the road nearer to the northern side of the opening and allow for an asymmetrical right-of-way.

56. Location of a Route

The specifications for a route project indicate the initial, the final and some intermediary points through which the route is to be laid. These are called *control points*, and may be industrial, administrative or cultural centres, or transport terminals. If one had to lay a highway along straight lines connecting the control points numerous obstacles would intervene, the negotiation of which would be less expedient technically and economically than their bypassing by deviation of the route from the straight line.

Basically there are two kinds of obstacles, namely, those *related to plan* and those *related to elevation*. The first type includes river bends, inhabited localities, sites with unfavourable soil and geological conditions, and reservations. Those of the second type comprise mountain ridges, cliffs, etc., deep or wide depressions, lakes and swamps.

The need to deviate the route from a direct alignment is governed by the control points through which the road is to be laid. These comprise established or determined railway crossings and intersections with roads of a higher class than the one being located, river crossings, mountain passes, and also stretches of existing roads which are to be incorporated.

As a result of survey and analysis of local conditions, on large scale contour maps a series of points are established, through which the possible routes can be located (bypassing obstacles).

By joining these points a number of alternative locations are produced.

Figure 116 shows several possible alternatives of bee lines between control points. On section *AB* the number of possible alternatives is determined by two saddles, *a* and *b*. The fixed points *c* and *d*, giving alternative bypassing of the lake, cause a further deviation of the northern route (continuous line on the figure). Small water courses discharging into the lake present no appreciable obstacle. The crossing of the next major obstacle—a large river—is possible

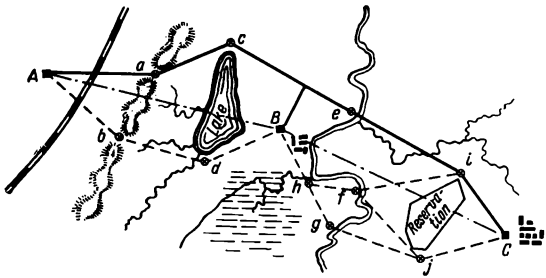


Fig. 116. Alternatives of bee lines between control points

at points *e*, *f* and *g* which offer the possibility of economical construction of a bridge. The location of these points indicates the expediency of selecting the northern route alternative, bypassing point *B* and extending to it an approach road.

For the southern alternative (broken line on the sketch) a tributary of the river and the swamped region in its upper reaches make it more expedient to route via point *B*, and from there to lead the route through the point *h*, bypassing the river bend. From there the alignment of both alternatives is determined by the boundary of a reservation (points *i* and *j*) which should not be crossed by the road.

The final choice between the southern and northern alternatives can be made only after an analysis of both projects and an appreciation of their transport and constructional characteristics.

The technical requirements which the horizontal and vertical constituent elements of any type of road or highway must comply with will inevitably cause further minor deviations of the route from the direct alignment. However, the comparison of profiles, plotted in accordance with contour maps and following the bee lines, enables a comparative appreciation of the two alternatives and the correct selection to be made.

The tracing of route lines on geological and soil maps also provides for the possibility of assessing the geological conditions for each alternative.

57. Crossing of Watercourses

A highway route will cross a great number of permanent and intermittent streams and rivers. The crossing of every watercourse almost inevitably leads to the erection of a structure for the discharge of water (bridges, culverts, percolation banks), of approaches to them (embankments), of control systems providing for the effective discharge of water through the structure, and to the necessity of protecting bridge abutments and the approaches to them from scouring.

Highway bridges and culverts should always be so arranged as to provide for free discharge at peak flow and comply with the requirements of economy of construction and convenience of traffic without hindrance to road continuity. In spite of the fact that the most economic and expedient watercourse crossing, from the viewpoint of water discharge, is that at right angles, skew crossings and crossings on a horizontal or vertical curve, or on a gradient are permitted when necessary. The higher the road class, the greater the justification for retention of the original route alignment instead of its diversion to permit the watercourse to be crossed at right angles.

On modern highways, in order to provide for the smooth horizontal and vertical alignment of the route, large bridges and viaducts may be constructed on horizontal and vertical curves (Figs. 117 and 118) notwithstanding the certain complexity resulting in bridge construction and in the building processes. Minor and medium bridges, and also culverts, may be located with any combination of vertical and horizontal elements that meet the requirements for roads of the given class.

Economic calculations indicate that a skew crossing of a small stream will increase the cost of a structure by about 20% when the angle of crossing is 60°, and by 50% when the angle is 45°. However, this extra expenditure is usually justified by the reduction of transportation costs, which, in the case of heavy traffic flow, would be increased considerably if the route were lengthened to make the crossing at right angles.

The members of bridges which are built along curves should comply with all the requirements which apply to the plan and cross-section of curvilinear road stretches, i.e., the insertion of superelevations, widenings and the introduction of transition curves, etc. The smooth alignment of the route should be combined with the choice of the crossing on a stretch having stable banks and bed, a parallel alignment of banks and a parallel flow of current.

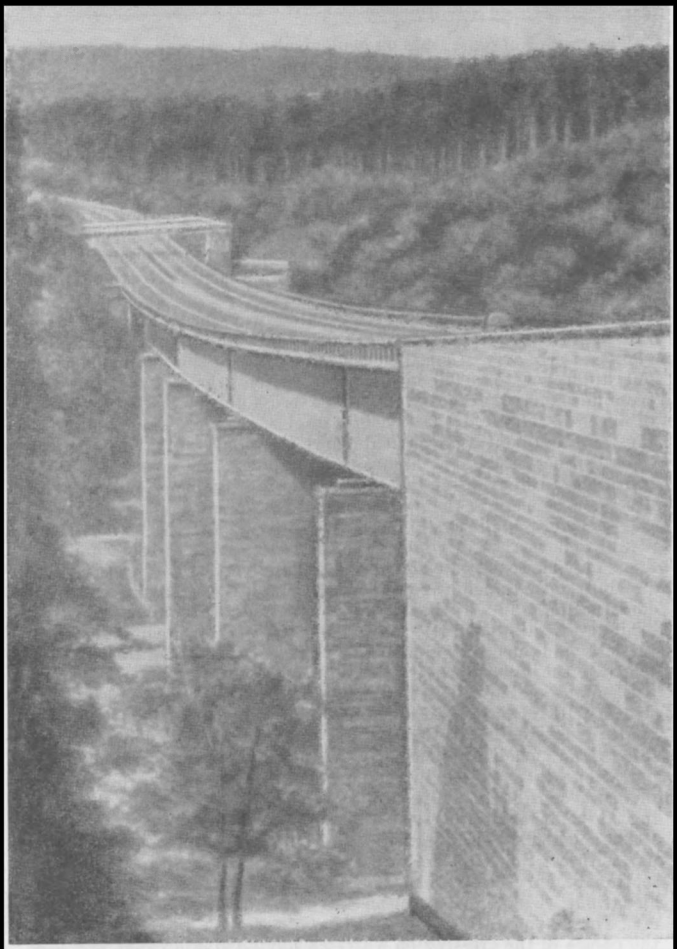


Fig. 117. Bridge on vertical curve

In the case of a route crossing a ravine at an angle the structure should be aligned normal to the axis of the ravine and at an angle to the route centre line. If the waterway axis is not parallel to the

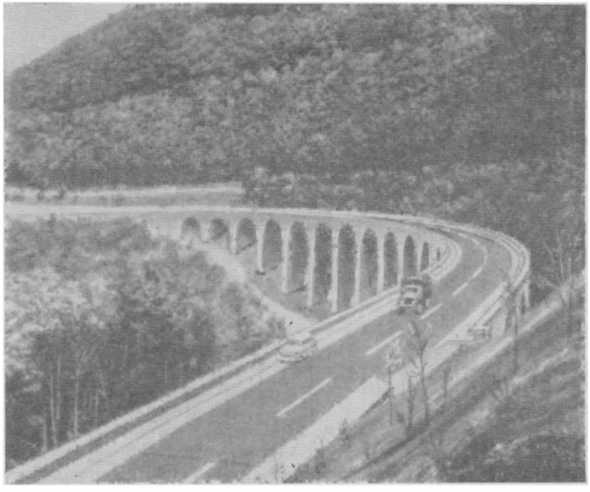


Fig. 118. Viaduct on curve

axis of the valley, it is expedient to straighten the river bed thus enabling all the construction work to be carried out in a trench excavated on dry land.

At crossings of major watercourses the construction of a skew bridge will increase appreciably the cost of the crossing and is coupled with the necessity for providing special control structures. The cost of a linear metre of a bridge over a major river is many times greater than that of a metre of an ordinary road. A major bridge, as a capital engineering construction, should be built on a stretch of river most convenient for the discharge of high water. This will provide for stability, prevent erosion of the bed, and comply with navigational requirements.

The convenience of a crossing from the point of view of motor traffic is secured mainly by constructing bridge approaches with a shallow gradient.

The selection of a crossing point is in most instances connected with a deviation from the desired alignment, and therefore the location of a bridge crossing over a major river should be considered as an essential route control point.

58. Route Development on Slopes

When locating a route in mountainous regions or a hilly country, and especially along river valleys and ravines, one frequently comes

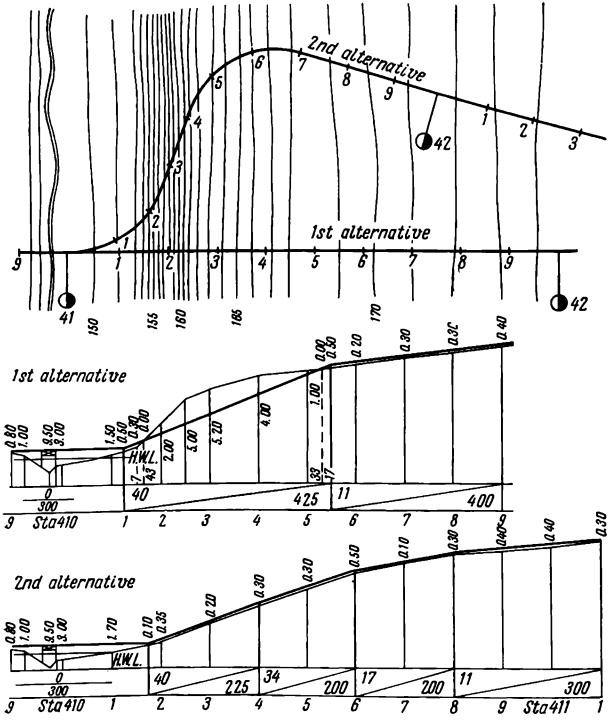


Fig. 119. Alternative road location on a hillside

across ground having slopes greater than the maximum allowable gradient on the road. In such cases two alternative route locations are possible (Fig. 119). The first alternative is a direct upgrade across the maximum hillside slope, which requires cutting through the hillside; the second is a rise following the hillside slope with a deviation from the bee line and with such an increase of the route length whereupon the longitudinal gradient will correspond to the limiting one for roads of the given class (*development of route along a slope*).

The required length of a hillside stretch is

$$L = \frac{100H}{i_{limit}} \tag{202}$$

where H = height to be surmounted, m

i_{limit} = limiting longitudinal gradient, %.

The negotiation of slopes along the shortest distance necessitates the building of embankments and cuttings, requiring extensive earthworks. However, in this case the road length is minimum, and this facilitates traffic if the gradient does not exceed 4 to 5%.

Alignment of the route to follow the natural slope usually reduces earthwork costs but leads to an increase in the cost of pavement construction, which partly absorbs the economy realized by earthwork reduction. The length of vehicle run is increased and the savings realized in road construction will be absorbed in time by the increase of vehicle operation costs.

For roads of lower classes with intermediate types of road pavements and having a small traffic intensity, the alternative of route development is more expedient. On the other hand, for highways of higher classes the advantages attained by reducing the length of the route are indisputable, and development of the route is practised only in cases when the depth of the cut or the height of the embankment becomes inadmissible for engineering reasons.

59. Route Location in Inhabited Localities

The use of highways for both local and through traffic necessitates their connection with intermediary inhabited localities. In this case there arises the question of convenient access for the locally generated traffic and the provision of convenient road connections to various urban areas.

When locating a highway in the neighbourhood of a town or city the problem arises as to whether the route should pass through the central area, or skirt the town altogether and be connected to it by an approach road. The choice depends on the size and importance of the town and on the relative proportions of through and local traffic.

When highways of classes I-III are located near small villages it is advisable that these be bypassed.

The increase of traffic intensity as a result of diverting essentially through traffic into a town will cause obstruction to local traffic. A road carrying heavy traffic divides the inhabited locality into isolated parts, influencing adversely its economic life. An increase in accidents may be anticipated together with increases in traffic noise and air pollution. The speed of through traffic is reduced considerably when passing through a built-up area and road maintenance becomes more difficult, especially the clearing of snow.

However, in cases when the through and local traffic within the limits of the inhabited locality are modest, the building of a first-class road may be considered as a temporary measure to accelerate the growth rate of the urban area. At the same time an alternative route bypassing the town should be envisaged, which is brought into commission when the traffic intensity increases.

The problem of locating a route in the vicinity of large cities is the most difficult to solve. Traffic investigations in some countries have shown that the relative importance of through traffic decreases with the increase in size of the city. The relation between the relative importance of through traffic and the city population, according to investigations in the U.S.A. and F.R.G. is as follows:

Urban population in thousands	1,000-500	500-300	300-100	100-50	50-25	25-10	10-5	<5
Through traffic, per cent	1	14	16	—	27	33	38	58

Parallel with this problem of diverting through traffic away from cities there arises the equally difficult problem of catering for the intensive traffic flow between the suburbs and the city.

The problem of route location through a large city is a function of the city layout, the contours of the existing road network, the situation of industrial enterprises, and of administrative, cultural and economic considerations. The location of the route may provide for intersections of main streets, roads through the outskirts or tangent roads flanking the boundary of the developed area. The last solution is the most expedient one, since it combines the facilities of communication with the city and the elimination of nuisance to the inhabitants caused by through traffic.

A city with dense traffic may be the focal point of several major traffic arteries. To eliminate through traffic it may be necessary to construct a separate bypass *belt road* without city limits.

The belt road is frequently laid adjacent to the boundary of the city development territory. In this case the belt roads will not

only improve conditions for the through traffic, but also facilitate internal urban traffic between the city outskirts by relieving its central districts of these vehicles. The belt road is connected to the city street network by means of access or approach roads.

In large cities, and also in industrial districts including residential areas and industrial buildings, situated at a distance of 20 to 30 km from the city centre, travelling will require an appreciable loss of time. To improve communication between the central districts and suburbs, in a number of American and West European cities limited-access high-speed roads (expressways) have been built carrying traffic flows from one district to another or from the outskirts to the centre, these roads being isolated from the remainder of the urban network. Vehicles can travel along these urban expressways at speeds similar to those on rural roads. Segregation of traffic is obtained by constructing the road as a flyover expressway, or in a tunnel or cutting, linking it to the main network with flyover crossings (see Figs. 240 and 241). The cost of such roads is very considerable, and their building necessitates demolition of an appreciable number of buildings and the erection of very involved engineering structures for traffic interchange at different levels.

60. Highway Intersections

Grade intersections of highways or of a highway and a railway are more congested than the stretches of highway between intersections, since the traffic intensity at a crossing is the total intensity of the two crossroads. With a comparatively low traffic intensity a system of traffic control is introduced at such crossings, which gives priority first to one road and then to the other.

Signal-controlled road crossings, which are widely used in urban conditions and at railway grade crossings, reduce appreciably the capacity of highways. Investigations show that a driver even when approaching a crossing giving him right of way will reduce speed, in case it may be necessary for him to stop before he reaches the control signal (traffic lights). Such a system is inadequate for high-speed roads carrying heavy flows, for which high-capacity grade separations are essential.

In accordance with the BS and R of 1962, grade separations must be provided in the following cases:

(a) at intersections of class I roads with all roads,

(b) at intersections of class II roads with those of classes II and III.

The simplest way of preventing the direct crossing of traffic flows is by the insertion of a traffic island or roundabout which compels the traffic to merge into one stream flowing round an island which is situated in the centre of the intersection. The radius of

the circle along which the traffic flows should correspond to a set speed at the crossing. Moreover, the radius of the circle should be sufficient to provide weaving sections of adequate capacity to permit the free regrouping of vehicles entering the roundabout flow and allow them to leave at the desired exit.

At a traffic speed of 40 km/hr the minimum radius of the island should be 25 m, at 60 km/hr—70 m, and at 70 km/hr—100 m. Owing to the regrouping of vehicles occurring at a roundabout their traffic speed is reduced.

The width of a weaving section of a roundabout depends on the number of lanes on the roads entering the roundabout and should be determined in accordance with the volume of traffic passing through the section. In Great Britain the diameter of the island D and the width of the roundabout carriageway B are related to the traffic intensity N as follows: with $N = 250$ veh/hr $D = 18$ m and $B = 9$ m; with $N = 400$ veh/hr $D = 31.5$ m and $B = 12$ m; with $N = 6{,}000$ veh/hr $D = 72$ m and $B = 15$ m.

On highways with heavy traffic flows flyover crossings are used, with one road crossing over another via an overpass. In this case through traffic may flow without appreciable reducing its speed, but the vehicles using the access roads to interchange from one road to another will have to reduce speed slightly.

In highway planning practice *clover-leaf* and *flyover roundabout grade separations* are most popular for right-angle intersections, while for acute-angle ones *direct-connection interchanges* (Fig. 120) are usually more convenient.

The most popular form of grade separation is the clover-leaf (Fig. 121*a*). In the G.D.R. and F.R.G. it is considered that such interchanges should be constructed at all highway intersections with roads of a lower class, when the traffic intensity on the highway exceeds 1,200 passenger cars or 800 trucks an hour. For interchanging between two highways separate direct or indirect connections are provided for each stream of traffic.

For connecting highways to roads of lower classes simplified half-clover-leaf interchanges are constructed.

To permit vehicles to drive off the highway and pass onto the exit ramp at a reduced speed, and also to allow vehicles coming off the access ramp onto the main highway to accelerate beforehand in order not to obstruct the main traffic flow, at the approach to the interchange additional tapered traffic lanes are constructed which are separated from the main carriageway by lines marked on the pavement. These additional lanes are called *acceleration* and *deceleration lanes*. The length of these lanes is determined in accordance with vehicle acceleration and braking characteristics. In the G.D.R. and F.R.G. the acceleration and deceleration lanes

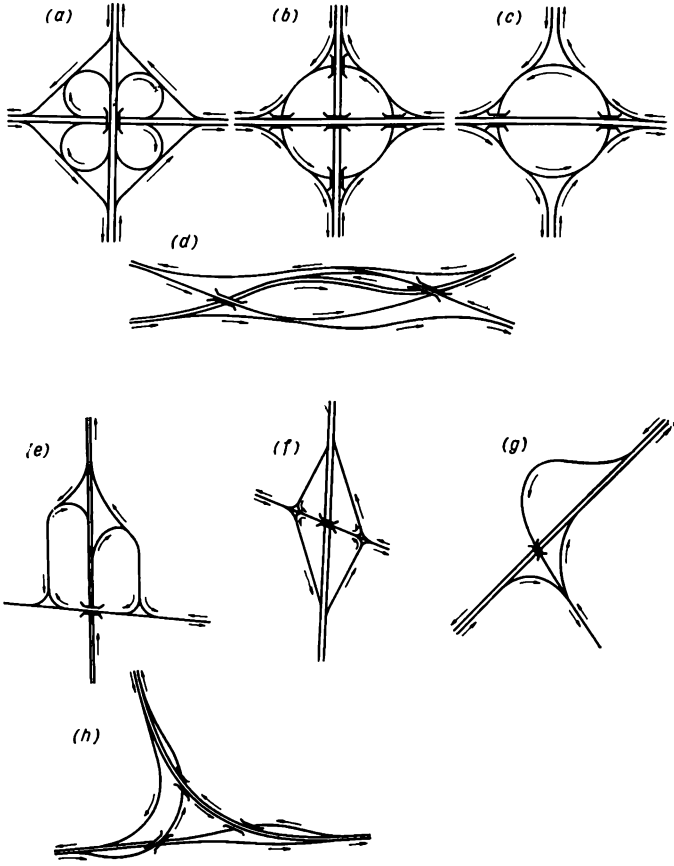


Fig. 120. Various types of interchanges and junctions:
a—clover-leaf; *b*—flyover roundabout; *c*—simplified flyover roundabout; *d*—direct-connection interchange; *e* and *f*—simplified interchanges of highway with road of lower class; *g*—trumpet junction; *h*—two-level junction

a)

b)

are 200 m long, of which 80 m are taken up by the tapered section abutting the main carriageway.

A flyover or bridged roundabout interchange may offer better facilities for turning traffic as it is possible to provide the roundabout with a greater radius than the ramps of a clover-leaf interchange. However, the cost of this interchange is higher since it is necessary to build five overpasses (Figs. 120*b* and 121*b*).

At crossings of motorways with roads of a lower class the interchanges can be of the diamond type, where vehicles joining the highway, or coming from it, will cross opposing traffic flows when making left turns (Fig. 120*f*).

In urban conditions, when designing expressways through areas with an appreciable street traffic, very complex crossings have to be provided for the interchange of traffic. The design of these interchanges has to be individually tailored to suit the special features of the traffic flows and site and of city developments.

The branches of highways are generally made according to the half-clover-leaf, trumpet or Y-junction design (Fig. 122).

Flyover interchanges are very complicated and costly structures, the design of which requires thorough engineering and economic study. To accommodate the ramps which may cross at several levels an extensive area is taken up by the interchange, while the total length of their slip roads may be several kilometres. Therefore, in order to reduce the overall dimensions of the interchanges, some reduction of vehicle speed is necessary—for those making left turns the reduction is about 70-75% of the design speed of the unrestricted sections.

In the U.S.S.R. it is customary to design the components of an interchange on the basis of the speed of goods vehicles. Therefore, designs are based on approach speeds to traffic interchanges along the nearside lane of at least 80 km/hr for roads of classes I and II and at least 60 km/hr for roads of class III. The radii of left-turn slip roads are designed for speeds, respectively, of 60 and 40 km/hr.

To economize on the cost of the interchange, the width of the pavement on ramps is limited to one traffic lane with a sufficient width of shoulders to accommodate parked vehicles. It is assumed that for this purpose a carriageway width of 4.5 m is adequate. The shoulders on the slip roads should be at least 1.5 m wide on the inner side and 3.0 m on the outer one. The shoulders are stabilized over a width of 75 to 50 cm.

Grade crossings of roads with railways are sited on straight sections. The vehicle driver should be able to see the on-coming train at a distance from the crossing equal to the design sight distance for the given class of road when a train is 400 m from the crossing. The locomotive driver should be able to see the crossing

Fig. 122. Highway junctions:
a—trumpet junction; *b*—two-level Y junction

at a distance of at least 1 km. When necessary, special measures are taken to ensure adequate visibility. The crossings should be situated outside of yard track limits. The angle of intersection should be as near as possible to right angles, but in any case not less than 45°. Depending on the traffic intensity on the road and the railway various types of crossings can be erected, which may be at grade or separated. Expressways and railways should always cross at different levels. For other roads the type of crossing is chosen in accordance with the intensity of automobile and railway traffic and with the regulations issued by the Ministry of Railway Transport.

61. Influence of Vehicle Requirements on Route Location

The requirements dealt with above, which horizontal and vertical road components are to comply with, were based on traffic safety considerations and the convenience of movement on each of these components considered separately. Actually the road is a combination of various sections on which the traffic travels with varying speeds. These comprise gradients, horizontal curves with a limited visibility and, finally, stretches where speed restrictions are imposed by traffic safety regulations (railway crossings, roads within inhabited localities, etc.). On short stretches with variable grad ents vehicles may not be able to travel at speeds corresponding to their rated dynamic characteristics and, owing to the frequent alteration of operating conditions, the engine power may not be used to the full extent.

The full utilization of the dynamic haracteristics of the vehicles should be envisaged at the design stage for roads of any class. The choice of route layout should provide for the possibility of high-speed movement. For this purpose, the number and extent of places which require reduction of speed for safety reasons should be reduced to a minimum (viz., places with limited visibility, combinations of steep gradients and bends, etc.). It should be noted that the *design speed* for each class of road relates to those stretches which are the most difficult and dangerous for the traffic, but for the res of the road provision should be made for a faster traffic speed. It the road profile one should avoid a succession of alternating sharn dips and crests, calling for additional power consumption. The psychological effect of road conditions on the driver, which determines the vehicle movement on the road, is most important and, unfortunately, has not yet been sufficiently investigated.

The methods of substantiating the requirements which the horizontal and vertical road components have to comply with and which have been dealt with in the first part of the book concern

the most difficult stretches of the road. These are based mainly on the need to provide for traffic safety in difficult driving conditions, with full use of braking power, limited time for driver's reaction, etc.

The investigation of vehicle operating conditions on roads has shown that the majority of drivers travel at speeds considerably slower than those envisaged in the design. This underlines the need for research into traffic movement on the roads, which is at present being carried out in a number of countries, in order to determine operating conditions convenient for drivers in the form of formulas for road component calculations.

The assessment by the driver of road conditions is determined by three factors: road and roadside visibility, traffic intensity, and the action of the dynamic forces which he is experiencing (inertia and radial acceleration, as well as impacts transmitted through the vehicle). For confident driving the driver must be able to envisage a section of the road ahead, the length of which is appreciably longer than the design braking distances used for determining visibility distances.

On roads whose geometric features do not conform to driving requirements, drivers will frequently unconsciously correct the designer's mistakes by cutting across curves of small radii, reducing speed on curves of a radius less than 600 m, etc., thus altering the road alignment to correspond to the values of the lateral force factor convenient for themselves.

In order that the road should best satisfy the requirements of traffic, its design should not adversely influence the driver's control of his vehicle, but should orientate the driver so that he can see the route alignment at distances appreciably exceeding the standard visibility distances. This is achieved by the tendency to provide horizontal curves of constant radii for long road stretches, since this tends to ensure uniform traffic conditions for the driver.

In this respect uniform application of curvature and standardization of road curvature are most important. The statistics of accidents show that on roads with a great number of curves of uniformly small radii fewer accidents occur than on a road having one curve of small radius among a series of curves of large radii. This is why in the design of some foreign highways the requirement was introduced that for a constant value of the lateral force factor μ the design traffic speed on adjacent route stretches (horizontal and vertical curves) should not differ by more than 9 to 13 km/hr. This is due to the fact that the driver becomes accustomed to road traffic conditions and, when these alter sharply, he does not immediately change the vehicle speed. Therefore, when designing roads laid through a varied topography, it is sometimes necessary, in conform-

ity with traffic safety regulations, to increase the cost of road construction over certain adverse sections, in order to comply throughout with the minimum geometric standards which were established for locations having a more favourable topography.

The driver's alertness depends on the road conditions. Under strenuous driving conditions he becomes more alert and reduces speed. This is confirmed by statistical data concerning the relative number of accidents in relation to the frequency of occurrence of road stretches with a limited visibility.

The combination of successive geometric road elements should contribute to traffic safety. The majority of accidents occur in places where the driver has to alter rapidly the dynamic conditions of vehicle control. As an example of sections prejudicial to traffic safety, which should be avoided in the planning stages, are places where:

(1) there is a sharp change in direction to be overcome by a vehicle moving at high speed, e.g., a curve of small radius located at the bottom of a long slope;

(2) there is a succession of hairpin bends which are negotiated at relatively low speeds and require frequent and rapid turning of the steering wheel, e.g., when a road follows the contours along a winding river valley with steep slopes;

(3) there are especially important crossings, stretches of road in populated areas combined with sections having poor horizontal and vertical alignment;

(4) there is a succession of crossing and merging traffic flows moving with various speeds, e.g., of through and local traffic.

62. Locating a Highway as an Integral Part of the General Landscape (Landscaping)

A modern high-class highway is a major public utility designed for lengthy service. Travelling on highways gives the opportunity to become acquainted with the country's scenery, landscape, its historical monuments, etc. In recent years many people have spent their holidays motoring. This is why the design of motor roads—especially those of higher classes—should be integrated with the surrounding natural landscape and blend in with the architectural structures situated along the road.

The coordination of a highway location and the surrounding country should take into account the importance of the road and the region of its construction.

Roads built in sparsely populated regions serve as an axis around which the settlement of the adjacent districts commences. The constructed road is designed for the economic assimilation of the

adjacent countryside. Such a road may be permitted to stand out prominently against the background with its geometrically regular contours of embankments and cuttings and its long stretches of regular form, and thus justifiably dominate the surrounding landscape.

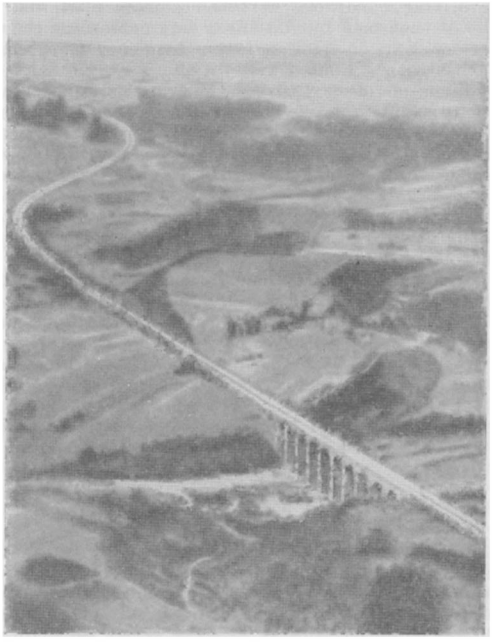


Fig. 123. General view of a highway following the land topography

On the other hand, in densely populated regions every effort should be made to harmonize the highway alignment with the natural landscape, and to emphasize or bring out the beauty of districts which remain unscathed during the construction period. In this case the fundamentals of landscaping demand that the road be perceived as a dimensional curve smoothly inscribed in the

natural forms of the landscape without, however, following in detail the small irregularities (Fig. 123).

Near holiday resorts the location of the road may be mainly aimed at displaying and emphasizing beautiful scenery.

A smooth road location is attained by the rational combination of its horizontal and vertical elements with the character of the relief. The smoothness of the route as a dimensional curve can be greatly promoted by combining horizontal and vertical curves having

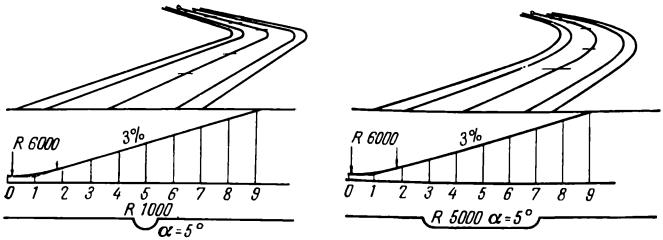


Fig. 124. Perspective view of a road with a bend of small and large radius

as large a radius as practicable. However, the size of the curve radii must be in scale with the length of the adjoining straight stretches. A short straight stretch between two adjacent curves presents an unpleasant break in the alignment. Such insertions should be replaced by curves of large radii or a compound curve. Straight stretches between reversed curves are replaced by connecting transition curves.

When crossing forests it is expedient to arrange for the entry and exit to be sited on curves in order to avoid an unpleasant view of a through cutting.

Long straights can have a tiring effect on drivers and should be periodically broken by the introduction of curves.

With an undulating topography the line of the route should be adapted to the character of the land. It is desirable that vertical curves of large radii be inscribed and interconnected, without the insertion of straights in the profile.

In order not to give the driver the impression of the road having a sharp bend, as it may appear to him in a perspective distortion even when the bend has a sufficiently large horizontal radius of curvature (Fig. 124), *special transition curves* are frequently provided on highways, the purpose of which is exclusively to give the route a flowing appearance. In this case the transition curve ceases to be

an auxiliary element of a curve of small radius, but becomes an independent element of alignment, just as important as vertical and horizontal curves.

During the last few years in some countries the principle of what is known as *optical alignment* has been widely introduced in the design of highways. This method consists in so locating a route as to make its direction clear to the driver for a long distance, substantially exceeding the minimum sight distance allowed by

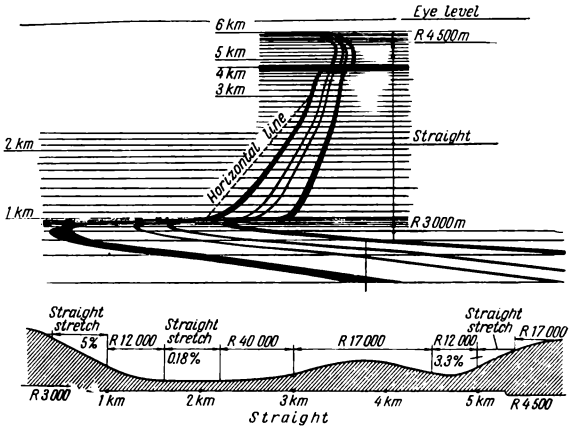


Fig. 125. Appraisal of route alternatives by using perspective view method

technical considerations. On long straight stretches the road is orientated on an outstanding object, i.e., a hill, a group of buildings, and in an open plain on a copse especially planted for this purpose. On convex vertical curves the continuation of the road alignment can be indicated by combining the vertical curve with a horizontal one which partly coincides with it in plan, or by planting high trees along the road, the tops of which indicate the route direction beyond the summit. Thus, the basic principle of optical alignment is to eliminate unforeseen changes in route direction.

For appraisal of the smoothness of the road and of the success of its blending with the land topography several alternative alignments are projected. For purposes of comparison, perspective views of these alternative route locations are drawn, showing the effects

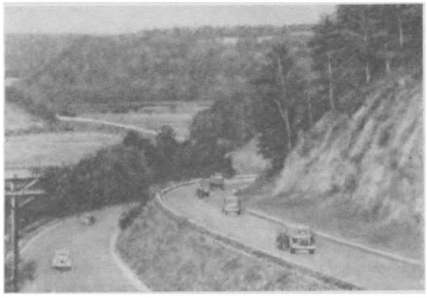


Fig. 126. Two-level arrangement of carriageways on a hillside

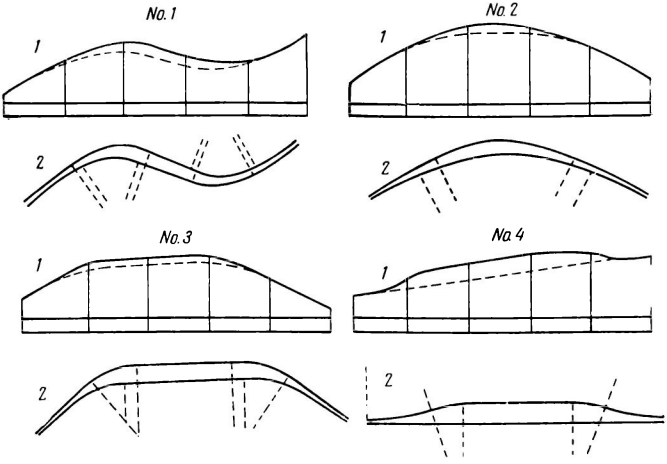


Fig. 127. Examples of designing road plans and profiles on stretches with a two-level arrangement of carriageways:
1—profile; 2—plan view

that will be produced both on the road itself and on the surrounding landscape (Fig. 125). In the case of difficult sections it is usual to construct scale models.

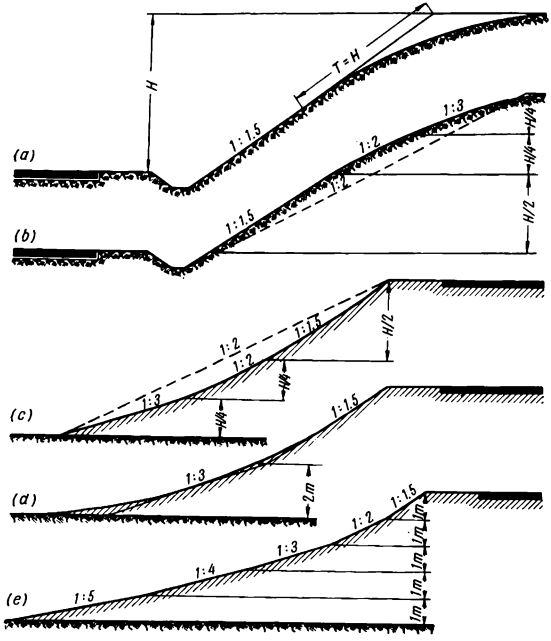


Fig. 128. Streamlined cross-sections:

a—cutting with slopes of a uniform gradient; *b*—cutting with cambered slopes, offering a streamlined surface to wind and snow; *c*—embankment with a diminishing slope gradient at its lower part; *d*—high embankment with reduced slope gradient along bottom 2 metres; *e*—embankment with gradually increasing slope gradient

When locating highways designed with a median in hilly country it is difficult to accommodate a wide roadbed without involving considerable cutting into the hillside, or the use of cross-sections of the cut-and-fill type. In this case it is preferable to divide the roadbed into two separate carriageways for each of the traffic streams,

which should be independently located at different levels on the hillside (a two-level arrangement of carriageways, Fig. 126).

A diagram illustrating the independent location of carriageways is shown in Fig. 127. The distance between the carriageways depends on the nature of the slopes.

In recent years, when designing highways through broken country, a new technique has appeared—the replacing of two carriageways on a single roadbed by two independent one-way roads, roughly parallel but 50 metres or more apart. Such a layout has the advantage of facilitating snow clearing and of eliminating the possibility of drivers being dazzled by the glare from on-coming head lamps at night.

Where there is a frequent alternation of comparatively low embankments and shallow cuttings the adaptation of the road to the landscape can be improved by employing streamlined cross-sections with slopes of variable curvature.

Gentle slopes are more stable and safer for traffic, and also adapt themselves better to the adjacent landscape than ordinary ones (Fig. 128).

To make the road harmonize with the landscape the existing vegetation should be preserved and extended by planting and seeding. Sometimes fully grown trees are preserved within the median. The width of the cut clearance in a tree belt should be reduced to a minimum, and by planting more trees and shrubs the impression of a geometrically rectangular opening is eliminated.

Turnouts are provided along a highway to enable vehicles to be parked for repairs, for relaxation of the drivers and passengers or other purposes. From the amenity viewpoint they should be located adjacent to spots with beautiful scenery, shady woods, springs with drinking water and near places where bathing is possible.

Turnouts are usually in the form of a widened lane, or a lane parallel to the main road.

CHAPTER 10

DESIGN OF PROFILE

63. Location of the Grade Line

When designing a road it is necessary, as a rule, to provide for some degree of elevation of the roadbed above the general ground level in order to ensure drainage of the subgrade and eliminate the numerous surface irregularities of a minor nature lying within the boundaries of the road. Roads are designed along the natural level of the ground only in exceptionally favourable soil and hydrologic conditions, namely, where the surface soils are permeable, in regions where there is no danger of snow drifts, etc. The construction of road embankments usually produces more favourable hydrologic conditions for the road pavement than the natural earth surface. Cuttings should be limited, as far as practicable, to comparatively short stretches where they are essential to reduce excessive longitudinal gradient and to decrease the quantity of earthworks.

The determination of the roadbed location in height is called the *design of the profile* or *location of the grade line.*

When locating the grade line it is necessary to provide for:

1. The preservation of a smooth flowing alignment which will enable vehicles to combine high speeds with safety.
2. The drainage of the carriageway, shoulders and subgrade beneath the road.
3. The location of the road through the fixed control points having specified elevations, i.e., junctions with existing roads at the terminals of the road, intersections with roads of higher class and with railways, points with preset levels of bridges, elevations of the roadbed above high water level in flooded areas, etc.
4. The reduction of the costs of construction and the easiness of mechanizing road construction work.

There are two methods for determining the location of a grade line—*envelope* and *intersecting design* (Fig. 129).

When using envelope design the tentative grade line is located as far as possible parallel to the ground surface, digressing from this rule only when crossing low-lying places or when a close succession of reverse gradients occurs. In flat or gently undulating country the method of envelope design combined with proper horizontal location and landscaping will give a well drained roadbed.

If the envelope design is used for locating the grade line in highly broken country the road profile becomes irregular (Fig. 129*a*),

vehicle movement on the road becomes a continuous negotiation of ascending and descending grades and leads to an excessive consumption of fuel and a reduction of traffic speed. With heavy traffic, the total cost of transportation will be substantially increased. In this case a more rational grade line would be one plotted using the intersecting method (Fig. 129*b*) by cutting through hills and employing the excavated material for erecting embankments in

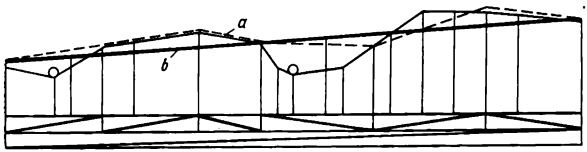


Fig. 129. Location of grade line along an enveloping curve (*a*), and along an intersecting line (*b*)

low places. The location of the grade line should, as far as possible in this instance, ensure the balancing of earthworks between adjacent embankments and cuttings, thus limiting the amount of soil to be borrowed from the side of the road.

In practice, both methods of locating the formation line can be used for matching the road alignment to the topography.

64. Design of Vertical Curves

The breaks in the profile which occur where there is a change in the gradient constitute a hazard to traffic: the convexities on the road reduce visibility; if the break is sharp the vehicle receives an impact when it crosses the crest of this irregularity; on convexities of small radius there is a danger of the vehicle leaving the road at high speed due to radial acceleration, or of loss of steering control owing to the sudden loss of weight on the forward axle; on concavities, owing to a sudden change of direction, an impact is induced which is disagreeable to the passengers and tends to overload the vehicle suspension. For this reason breaks in the profile are smoothed out by the introduction of connecting *vertical curves* (Fig. 130). These curves are obligatory with an algebraic difference of the longitudinal gradients of 0.5% and more on roads of classes I, II and III, and of 1% and more on roads of classes IV and V. The breaks in the profile which are thus alleviated are shown in Fig. 130 by a dotted line. The figures in brackets specify the ground and grade elevation differences without taking into account the

vertical curves, and the figures without brackets are the actual elevations. The radii of the vertical curve inserts are determined by the need to ensure for the vehicle driver a sight distance adequate for an emergency stop. The fulfilment of this requirement provides

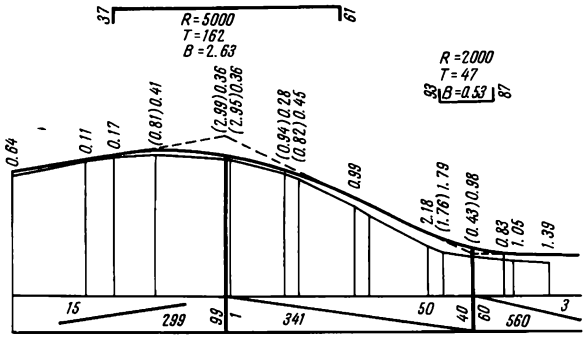


Fig. 130. Vertical curves

for the safety and convenience of the traffic. The calculation is based on simple geometric relationships (Fig. 131).

Let a_1 be the height of the driver's eye above the road surface and a_2 the height of the obstruction which should be visible. According to Fig. 131 the design sight distance on a convex curve consists of the sum of two arcs l_1 and l_2. From the similarity of the triangles ABC and ACD we find

$$\frac{BC}{AC}=\frac{AC}{CD} \quad \text{or} \quad BC\times CD=AC^2 \tag{203}$$

Since the radii of the vertical curves are very much greater than the values of a_1 and a_2, one can assume, without substantial error, that

$$BC\approx a_1,\ CD=(2R-a_1)\approx 2R, \text{ and } AC\approx l_1$$

Introducing these values into the equation (203), we obtain

$$l_1=\sqrt{2a_1R}$$

Similarly we can determine $l_2=\sqrt{2a_2R}$.
Therefore, the design sight distance is

$$L=l_1+l_2=\sqrt{2R}\left(\sqrt{a_1}+\sqrt{a_2}\right)$$

Solving this expression in respect to R, we obtain

$$R=\frac{L^2}{2(\sqrt{a_1}+\sqrt{a_2})^2} \tag{204}$$

If the calculation is made for two opposing vehicles of the same type, then, neglecting the difference between the eye levels of

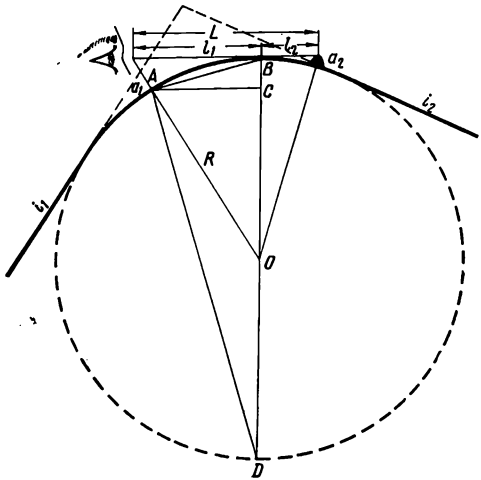


Fig. 131. To calculation of vertical curve radii based on sight distance requirements

the drivers and the heights of the vehicles, we obtain

$$R=\frac{L^2}{8a} \tag{205}$$

If the calculation is made for the visibility of the road surface we have $a_2 = 0$ and, therefore,

$$R=\frac{L^2}{2a} \tag{206}$$

The determination of the radius of concave curves is based on the centrifugal force which is acceptable for passengers and for the overloading of springs. The tolerated overloading can be expressed

in relation to the loading of a wheel

$$Q = kG$$

where G = pressure on the wheel due to the vehicle weight
k = coefficient of additional loading, usually accepted as equal to 0.05-0.10.

Using the expression for the centrifugal force on a vertical curve, we obtain

$$\frac{Gv^2}{Rg} = kG$$

Hence

$$R = \frac{v^2}{gk} \tag{207}$$

On roads having a design traffic speed of 120 km/hr the radii of convex vertical curves should not be less than of 10,000 m, and of concave ones 5,000 m.

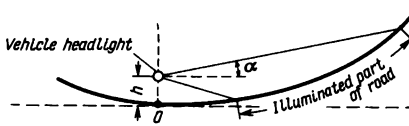


Fig. 132. To determination of concave vertical curve radius

At night the head lights illuminate only a part of the concave curvature, which is the lesser, the smaller the curve radius. The radius of concave curves necessary for the provision of an adequate sight distance at night time can be determined from the following considerations (Fig. 132).

Placing the origin of the coordinates at point O, let us replace the equation of a circular vertical curve by an approximative parabolic one

$$y = \frac{x^2}{2R}$$

which is obtained from an approximate relation between the radius, the chord and the deflection of the segment.

When the head lights are at a height h above the road surface, the equation of the upper level of head lights illumination will be

$$y = h + x \tan \alpha$$

where α is the angle of vertical light distribution in a vertical plane, which is usually assumed to be 10°.

The expression for determining the distance at which this light beam intersects the pavement surface is obtained by equating the equations for y

$$\frac{x^2}{2} = h + x \tan \alpha$$

The radius of the curve necessary for ensuring visibility corresponds to the case when the abscissa x is equal to the design sight distance S of the road

$$R = \frac{S}{2\left(\frac{h}{S} + \tan \alpha\right)} \approx \frac{S}{2 \tan \alpha}$$

To provide adequate visibility at night the radii of concave vertical curves should be from 0.35 to 0.5 of the convex curve radii.

In the case of circular concave curves the overloading due to radial acceleration appears immediately on entering the curve. To avoid this it is necessary to introduce transition curves in the profile or alternatively use a cycloid type of concave curve in which the curvature increases gradually and attains the maximum value in the middle of the vertical curve.

The minimum radii of vertical curves for roads of various classes are presented in Table 28.

The BS and R recommend, when this is not connected with an increase in costs, to always use in designing roads:

Radii of vertical convex curves of at least 60,000 m
Radii of vertical concave curves of at least 8,000 m
Lengths of vertical convex curves of at least 300 m
Lengths of vertical concave curves of at least 100 m

Tables can be compiled for determining the elements of vertical curves using the same formulas employed for horizontal curves. However, since the vertical angle between the elements of the grade line is small with large vertical curve radii, simplified formulas are used for laying out vertical curves.

The angle α of the change of direction in the profile, expressed in radians, is equal to

$$\alpha = \Delta_1 + \Delta_2 \approx \tan \Delta_1 + \tan \Delta_2 \approx i_1 + i_2$$

where Δ_1 and Δ_2 are the respective angles formed by the grade line with a horizontal plane.

The length of the tangent of a vertical curve is

$$T = R \tan \frac{i_1 + i_2}{2} \tag{208}$$

TABLE 28

Curves	Radii of vertical curves (in metres) depending on road class				
	I	II	III	IV	V
(a) Convex:					
Level country	25,000	15,000	10,000	5,000	2,500
Difficult sections:					
Broken country	15,000	10,000	5,000	2,500	1,000
Mountainous country	5,000	2,500	1,500	1,000	600
(b) Concave:					
Level country	8,000	5,000	3,000	2,000	1,500
Difficult sections:					
Broken country	5,000	3,000	2,000	1,500	1,000
Mountainous country	2,000	1,500	1,200	1,000	600
Exceptionally difficult sections:					
Level country	4,000	2,500	1,500	1,000	600
Broken country	2,500	1,500	1,000	600	300
Mountainous country	1,000	600	400	300	200

The correction y in the elevation differences at a point B, situated at a distance x from the origin of the vertical curve, can be determined as follows (Fig. 133). If the point B is joined to the centre of

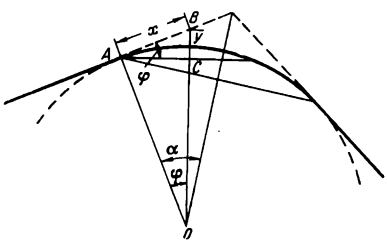


Fig. 133. Diagram showing coordinates of a vertical curve

the circular curve and a chord is drawn from the start of the curve or commencement point (point A) at right angles to the line OB, the value of the correction y can be assumed to be approximately

equal to half of the distance from the tangent to the chord

$$y = \frac{BC}{2} = \frac{1}{2} x \sin \varphi \approx \frac{1}{2} x \tan \varphi \tag{209}$$

where φ is the angle between the tangent and the chord.

However, $\tan \varphi = x/R$

whence

$$y \approx \frac{x^2}{2R} \tag{210}$$

With the values of longitudinal gradients and radii of vertical curves encountered in practice the error in determination of the correction is never greater than 5% of the value calculated according to precise formulas even in the most unfavourable cases, and is within the limits of the permissible accuracy in marking out the line.

65. Sequence of Designing the Profile

The design of the profile is commenced by locating the control elevation points and by establishing the required elevations of the pavement sub-base along the centre line of the road at various sections depending on the soil and hydrologic conditions. Next, the preliminary grade line is plotted, using patterns drawn to the scale of the profile that show the inclination on it of lines having differing longitudinal gradients and vertical curves of various radii. The plotting of the formation line should be aimed at arranging for the compensation of the volumes of adjacent embankments and cuttings. Since with equality of the elevation differences the cross-section of a cutting will be larger than that of an embankment, it is necessary to locate the formation line in such a way as to make the area of the sections with cuttings along the profile some 25-30% less than the area of embankments. The elevation differences should not exceed by more than 20-30 cm those required for local soil and hydrologic conditions.

When plotting the elevation of the formation line one should avoid frequent breaks corresponding to minor irregularities in the profile of the ground. It is particularly necessary to avoid any rapid alternations of grade (saw tooth profile). At the same time one should not introduce artificially long stretches with a constant gradient, the construction of which would require excessive earthworks and which would not be in harmony with the topography of the surrounding landscape.

The minimum acceptable length of a stretch with a uniform gradient is one which will permit the coincident location of the tangents of successive vertical curves. For ensuring a smooth grade line the distances between the apices of adjacent convex and con-

cave breaks in profile should be at least 200-300 m for high-class roads, and 100-150 m for low-class ones.

At present two methods for designing the profile are employed:

(1) drawing the grade line as a series of connecting straights and subsequently inscribing into the angles appropriate vertical curves, calculating the corrections to the elevation differences found along the tangents;

(2) drawing the vertical curves as they occur, either directly connected or joined by straights, and immediately calculating the grade line elevations.

The first method is more convenient for either flat or mountainous terrain, the second one is for highly broken hilly topography when for the major part of its length the road consists of a continuous series of vertical curves.

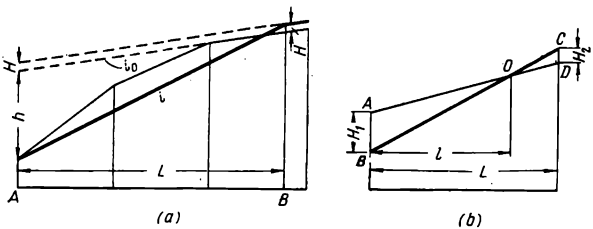


Fig. 134. Setting out problems:
a—determining place where line having a given profile emerges to surface;
b—determining point of change from a cutting to an embankment

When designing the grade line as a series of straights, after preliminarily plotting its location the grade elevations at the intersection points of the profile are calculated and the gradients are finalized by so changing the elevation differences as to express the gradient of the formation line in tenths of a per cent. Having coordinated the gradients and the elevations at the breaks of the grade line, the intermediary grade elevations and elevation differences are calculated and the vertical curves are inscribed. If the elevation differences are not convenient, and, for example, the intermediate grade elevations along the road centre line are not at an adequate height above the land surface or the water table, or the road is in a shallow cutting for a great length, the gradient will need to be amended, together with the initial elevation differences.

In the process of locating the grade line the following particular problems have to be solved:

1. *Determination of the place where the grade line having a given longitudinal gradient i intersects the ground line* (Fig. 134*a*). Initially the longitudinal gradient i_0 of the ground line is determined for the section where, judging from the profile, the grade line should emerge. Then a fictitious elevation h is computed for the continuation of this gradient to point A at the origin of the upgrade. Adding to this elevation the required elevation of the sub-base line along the centre line of the road at point B, the length of the upgrade section is determined according to the formula

$$L = \frac{100\,(h+H)}{i-i_0} \tag{211}$$

2. *Determination of the point of change from cutting to embankment* (Fig. 134*b*). The distance l from the origin of the section along which the grade line passes from a cutting onto an embankment (or the opposite) is obtained from the similarity of triangles ABO and OCD.

$$\frac{H_2}{L-l} = \frac{H_1}{l}$$

where H_2 and H_1 = elevation differences at the start and at the end of the section, metres

L = extent of the section within the limits of which the grade line and the ground line have constant gradients, metres.

Whence

$$l = \frac{H_1 L}{H_1 + H_2} \tag{212}$$

When the second method is used for design, i.e., the drawing of vertical curves, transparent patterns of vertical curves are superimposed on the profile, which are cut out according to its scale. Using a series of curve patterns of various radii, the most favourable location of the grade line is plotted graphically.

Along the perimeter of the patterns various points of contact of tangents corresponding to straights having various gradients are marked (Fig. 135). The alignments of these tangents permit plotting the longitudinal gradients of the straight stretches of the grade line adjoining the curve.

The levels of the vertical curves are calculated according to formulas derived from the diagram shown in Fig. 136:

1. With a known elevation at point A (designing from "left to right") the elevation of point F above it is

$$h_1 = DO - CO = R\,(\cos\alpha - \cos\beta) \tag{213}$$

The value of the angle β is determined according to the pattern depending on the chosen point of connection of the vertical curve

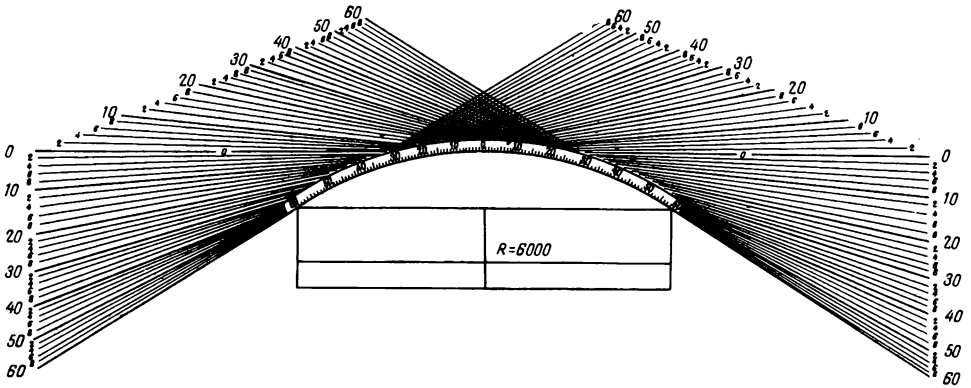


Fig. 135. Template for designing vertical curves

with the adjacent straight. The angle α can be determined from the relation

$$\sin \alpha = \frac{T - l_0}{R}$$

For the points situated on the right-hand half of the curve the computation is made in a similar way.

2. With a known elevation at point M (designing from the summit) its elevation above point F is

$$h_2 = MO - DO = R(l - \cos \alpha) \tag{214}$$

where α is determined from the relation $\sin \alpha = l/R$.

The development of computer technique makes possible designing of the profile for motor roads using electronic computers. This has

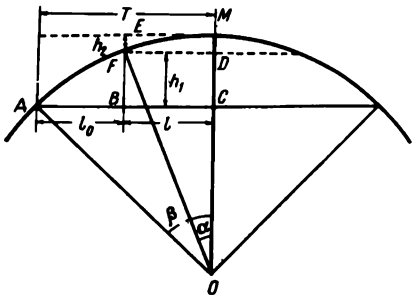


Fig. 136. To determining elevations of the formation at any point on a vertical curve

been successfully applied. The programming of electronic computers for this work required the availability of information concerning ground elevations along the road alignment and a system for computing the elements of the grade line taking into consideration the speed of the traffic and safety requirements. The electronic computer calculations gave a series of alternative grade lines according to various selected conditions. For each station the grade elevations and traffic speeds were determined.

66. Determination of Reference Points for Locating the Grade Line

A formation line should pass through a series of reference points or elevations. Some of these elevations will be strictly fixed, e.g., intersections of railways or existing highways at grade, and the

start and the end of the route. The location of other points of reference is determined in accordance with their required minimum elevation above ground level, which may be adjusted in order to ensure a flowing alignment of the grade line, though the latter entails an increase in construction costs. These points of reference include elevations above bridges and culverts, the elevation of the roadbed above flood line, etc.

The elevations of the grade line above the reference points should be set before designing the profile. The eievation of the roadbed

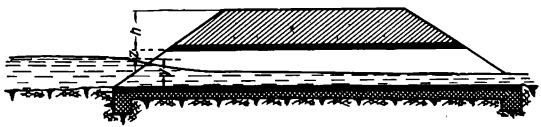


Fig. 137. Headroom of a structure above the lowered part of a channel

bottom along the road centre line at places of prolonged ponding of surface water and of high water table are determined according to Sec. 38 of this book.

In exposed places, where appreciable snow drift is likely, it will be good practice to elevate the roadbed bottom along the centre line of the road to 2-2.5 times the average thickness of the snowfall over many years, taking into consideration experience in the maintenance of existing highways and railways.

It is most difficult to establish the elevations and to plot the grade line on stretches adjacent to structures (bridges, etc.). Here the grade line should provide for a vertical clearance sufficient to permit unimpeded functioning of the structure during the period of high water. The location of the grade line at the approaches should ensure their safety against flooding.

The magnitude of the bridge clearance above the lowered part of the channel bed (Fig. 137) is made up of the following components:

(1) the depth of flowing water y taking into account the backwater at the entrance to the structure;

(2) the clearance between the backwater level and the lower part of the span z; for small structures the headroom should provide for safe passage of floating objects and avoid the danger of these structures being inundated during floods, the level of which exceeds the design one, and on navigable rivers to permit the passage of vessels;

(3) the height of the span, and for culverts also the thickness of the soil layer above the culvert, including the thickness of the culvert wall h.

On flood plains the elevation of the embankment is determined according to the backwater level. On big rivers, where wide areas are flooded during the flood season, and waves may be formed, the level of the embankment edge should be set according to the required margin above the height of the wave.

The embankments of the approaches are designed for floods occurring less frequently than the frequency of the bridge-design flood, which can be justified by the fact that bridges have a shorter life than the approach embankments. Embankments subject to overtopping in flood time may be built only for roads of lower class, but they should be thoroughly reinforced in order to avoid washout.

The minimum elevation of the embankment on the approaches to the bridge is determined by the design water level, the probable annual frequency being: for roads of classes I and II—1% (once in a hundred years); for roads of class III—2% (once in 50 years); for roads of classes IV and V—3% (once in 33 years).

The edge of the embankment should be above the accepted design water level plus the height of the backwater and the height of the wave allowing for a minimum surge of 0.5 m.

The elevation of the edges of berms and of unflooded regulating structures in identical conditions is taken equal to 0.25 m.

The height of the waves during river floods has not been investigated. Andreyanov's formula, which has been established following the observations of wave formation with a fetch of from 3 to 30 km, leads to a somewhat excessive margin of allowance

$$h = 0.0104 v^{5/4} L^{1/3} \tag{215}$$

where h = height of the wave above the still water level, m
v = wind velocity, m/sec
L = length of the fetch, km.

The height of the surge of the wave against the slope of the embankment, according to N. N. Djunkowsky's formula, is

$$h_s = 6.4kh \tan \alpha \tag{216}$$

where α = angle of the embankment slope above the water level
k = coefficient characterizing the roughness of the slope:
for a smooth surface (concrete slabs, level paving) $k = 1$,
for a rough surface (coarse rubble fill) $k = 0.77$.

For navigable rivers the bridge headroom is designed according to the estimated navigable level, which is somewhat lower than the peak flood level.

The vertical clearances are given in Table 29.

TABLE 29

Crossed watercourse	Vertical clearance	
	Design flood level	Other levels
Navigable and floatable rivers	From 1.5 to 13.5 m depending on the class of the river and in accordance with design standards of bridge headways for navigable and floatable rivers	—
Rivers not navigable and not floatable	0.25 m + wave height; when there is a possibility of obstruction forming on the river or mudflows—minimum 1 m, depending on the nature of the watercourse	Minimum 0.75 m above ice drift level
Blind creeks:		
(a) bridges	0.25 m	Above ground level: on roads of 1st and 2nd class—1 m; 3rd, 4th and 5th class—0.7 m
(b) culverts	1/4 of the culvert clear height, but not exceeding 0.5 m	

Since the bridge carriageway, when designed in accordance with the requirements for minimum earthworks at the approaches, will be higher than the carriageway of the approaches, a bulge is produced in the profile (hump-back bridge). A vehicle moving at high speed is subject to an abrupt radial acceleration at such a change of profile with a consequent reduction in the operational performance of the road. The need to provide uniformity of conditions for traffic, therefore, makes it necessary to increase the height of the adjacent embankment, so eliminating the hump.

When planning small structures, apart from this, a series of other techniques are employed to ensure a flowing alignment.

1. *Location of bridges with a longitudinal fall* (Fig. 138). When wooden floorings are used the bridge carriageway longitudinal gradient should not exceed 2 to 3%, depending on the way the boards are arranged. If one type of surfacing is used on the bridge and on the approaches, then the maximum longitudinal gradient of the bridge should be the same as for the approaches. Small bridges located on a valley slope are sometimes sited close to one of the sides and the stream diverted via an artificial channel under the bridge. The section with a longitudinal gradient along which a bridge is situated should be extended for some distance on both sides of the bridge, beyond the limits of the span.

2. *Location of the bridge along a vertical curve.* This solution avoids breaks of the formation line in the vicinity of the bridge, which are inevitable if the bridge is to be given a horizontal deck.

3. *Deepening of the river bed under the bridge.* This solution becomes necessary when crossing shallow thalwegs in a plain, in which the depth of the natural water flow does not exceed 20-30 cm. All

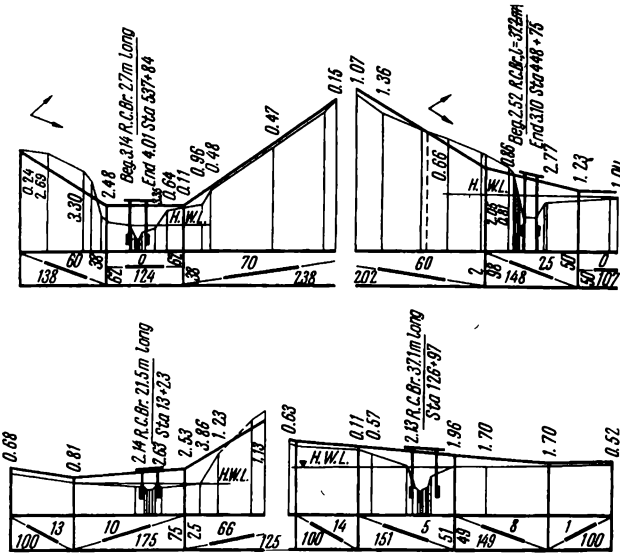


Fig. 138. Examples of plotting route line at minor bridges

the stream, or the major part of it, flows through a ditch under the bridge. To deepen the bed it is necessary that the slope at the crossing should enable the ditch to be given a longitudinal gradient sufficient to prevent silting of the bed, and that the ditch should be brought out to the surface near the bridge.

4. *Lowering of the carriageway level over bridges, etc.* This is obtained by reducing the design velocity of the water flow in order to reduce the depth of flow and the backwater function, or by replacing one culvert of a large diameter by several smaller ones having an equal total discharge capacity.

When planning embankments at crossings over narrow and deep ravines the carriageway level of the bridge, viaduct, etc., as determined by equalization of cut and fill earthworks, is often higher than is required by the minimum clearance to permit the discharge of flood waters. Since high bridges are built upon piers and the length of their spans increases substantially with an increase of their height, at crossings of narrow and deep ravines it is expedient to lay culverts (Fig. 139).

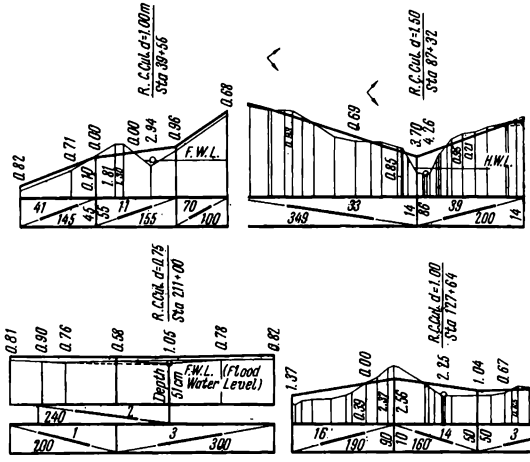


Fig. 139. Examples of plotting route line over culverts (for clarity the vertical curves are not shown)

At crossings of large navigable rivers some elevation of the bridge carriageway above the approaches is inevitable (Fig. 140), since otherwise the embankments would become very high. In this case the formation line should provide for an easy rise to the bridge. For this purpose the gradient of the approaches to the bridge should not exceed 3%, and between the end of the upgrade and the start of the bridge a horizontal section should be introduced of sufficient length to permit the location of the tangents of the vertical curves. On large bridges having flood spans the additional elevation necessary for the navigable part may be attained by designing these spans on a longitudinal gradient.

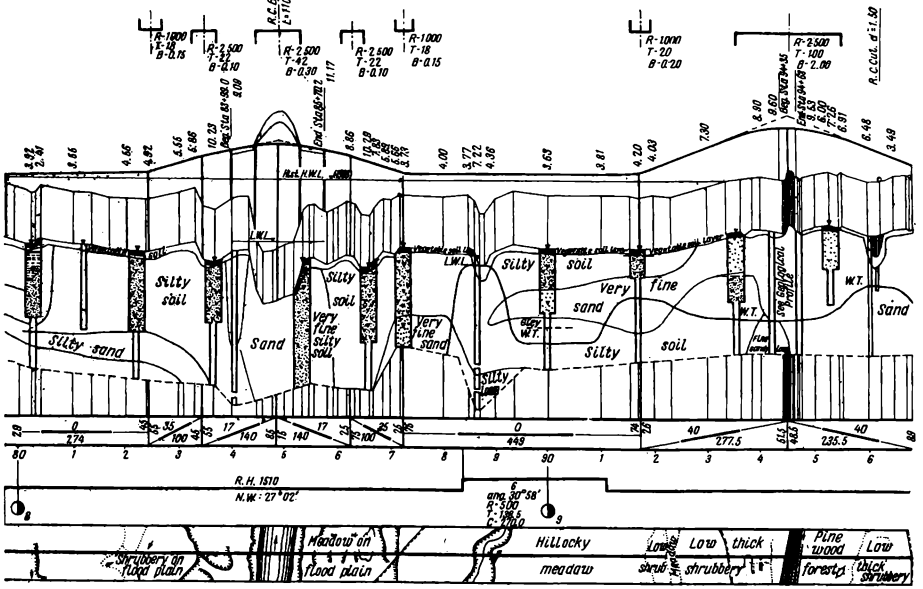


Fig. 140. Profile of a major bridge crossing

The location of the grade line should also provide for the continuity of water discharge from the road along the roadside ditches and borrow pits.

Since the ditches are sited parallel to the edge of the road their gradients are approximately equal to that of the highway. Along the whole stretch of each ditch section—from the watershed to the approach of the structure, or to the point of discharge from the ditches—the ditch should have a continuous fall sufficient to allow a free flow of water without ponding. For this reason roadside ditches which become overgrown and are used only periodically should receive a minimum gradient of 0.5%. Only in exceptional cases, in particularly difficult conditions of water diversion in plains, may a reduction of the longitudinal gradient to 0.2% be tolerated. Every opportunity of diverting water from the ditches away from the road should be used, arranging in suitable places diversion channels which are given a minimum slope of 0.2%.

To avoid the overflowing of the upper intercepting ditch in sidelong ground, where low spots occur in the profile, culverts of a convenient size are passed under the road in order to transfer water from the ditch on the upper side into the lower side ditch. It is desirable that the diversion of water from the side ditches away from the road or through a structure should be arranged at intervals of not more than 500 m.

If owing to the land relief it is impossible to collect water by means of side ditches, the road must be constructed on an embankment filled to a minimum height of 0.5 to 0.6 m. In this case the depth of the borrow pits at individual sections is fixed with the aim of providing for the possibility of diverting water longitudinally by giving their inverts a modest fall, which helps to prevent ponding.

On individual short horizontal sections, mainly near watersheds, the side ditches are made much deeper for the collection of water. These ditches are not made parallel to the edge of the road, they are also provided with a slight fall to assist the discharge of water. In this case the depth of the ditches is increased as they move away from the watershed. One should avoid an additional deepening of ditches of more than 0.6 m beyond their normal depth—which is determined by considerations of the soil and hydrologic conditions—because even with side slopes of 1 : 1.5 a ditch 1.0 to 1.2 m deep is about 3.5 to 4.0 m wide at the top.

67. Volumes of Embankments and Cuttings

For compilation of a program of work organization, the selection of the road machinery required and for the assessment of building costs the quantity of earthworks necessitated by the construction

of the individual road sections and of the road as a whole should be determined. The calculation of the earthwork quantities is based on the elevation differences marked along the profile.

A short stretch of a roadbed between two adjacent points of break in the profile where no transverse slope of land exists can be considered as being a regular geometric solid—a prismatoid with a trapezoidal base (Fig. 141).

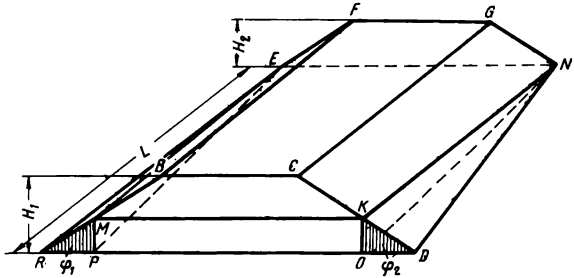


Fig. 141. Determination of volume of embankments and cuttings on level ground

To deduce the formula of the prismatoid volume a plane is inserted passing through the line *EN* and parallel to the upper surface of the prismatoid *BFGC*. Then, by simple transformations the prismatoid is divided into four regular geometric solids: a prism with a trapezoidal base *MBCK*, a wedge-shaped prism with a triangular base *KON* and two triangular pyramids—*KODN* and *MPRE*. The volume of these solids is

$$\left.\begin{aligned} V_{prism} &= \omega_{MBCK} L = F_2 L \\ V_{wedge} &= \omega_{MKPO} \frac{L}{2} = (F_1 - F_2 - \varphi_1 - \varphi_2) \frac{L}{2} \\ V_{pyramid} &= \varphi_1 \times \frac{L}{3} = \varphi_2 \frac{L}{3} \end{aligned}\right\} \qquad (217)$$

After summation of the resultant volumes and the necessary transformations, the formula of the prismatoid volume is obtained

$$V_{prismatoid} = \frac{F_1 + F_2}{2} L - \frac{\varphi}{3} L = \left[\frac{F_1 + F_2}{2} - \frac{(H_1 - H_2)^2 m}{6} \right] L \qquad (218)$$

where m is the side slope of the roadbed.

The elevation difference at the middle section is equal to half of the sum of the elevation differences at the end cross-sections

$$H_{mid} = \frac{H_1 + H_2}{2}$$

The half sum of the areas F_1 and F_2 may be expressed by the area F_0 at the middle cross-sections of the prismatoid with the height H_{mid}.

By means of algebraic transformations the equation (218) can be reduced to

$$V_{prismatoid} = \left[F_0 + \frac{m\,(H_1 - H_2)^2}{12} \right] L \qquad (219)$$

In equations (218) and (219) the second members are small in comparison with the first ones and need to be taken into consideration only when the elevations H_1 and H_2 differ by more than

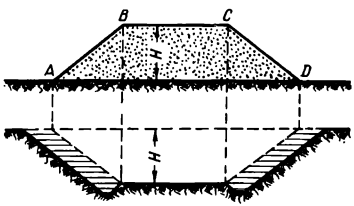


Fig. 142. Difference in volume of embankments and cuttings having same height

one metre. With a smaller difference between adjacent elevations simplified expressions can be used for determining the quantity of earthworks

$$V_{prismatoid} = \left(\frac{F_1 + F_2}{2} \right) L \qquad (220)$$

$$V_{prismatoid} = F_0 L \qquad (221)$$

The first of these equations gives a somewhat increased quantity of the earthworks, and the second one a slightly reduced one. The equations (220) and (221) are equally suitable for determining the volumes of embankments and cuttings. With equal elevation differences and widths of carriageway the volumes of cuttings exceed those of embankments due to the width of the side ditches extending to their top (Fig. 142).

68. Computation of Earthwork Quantities

To compute the earthwork quantities the design organizations make use of special tables which have been compiled for various widths of road according to equation (220). Usually the tables give earthwork quantities for various values of the sum of the elevation differences $H_1 + H_2$ for different section lengths L.

For convenience of calculation, the volume of side ditches is included in the volume of cuttings. For the computation of embankments the volume of side ditches is assessed according to special tables.

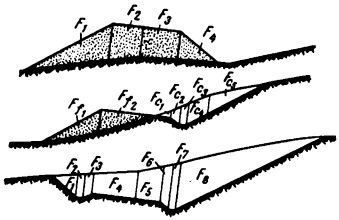


Fig. 143. Computation of earthwork quantities from cross-sections

A sidelong slope of less than 10% has an insignificant influence on the volume of earthworks and is not taken into consideration in the calculations. In steep sidelong ground the earthworks are computed according to equation (220). To facilitate computations, cross-sections of the roadbed should be drawn at characteristic points (Fig. 143). The areas of cuttings and fills are measured by means of a planimeter or by breaking up a complicated section into simple geometric figures.

Bridges having spans of less than 4 m and culverts are not taken into account for the purpose of computing earthwork quantities and are entered as if they were filled with earth.

For a more precise assessment of the earthwork quantities involved in road construction a correction has to be made to the quantities calculated according to the formulas which takes into account the following: the effect of variations in adjacent elevation differences if these exceed 1 m; the additional earthworks required for filling behind abutments at bridges; the volume taken up in a finished road by the pavement (carriageway correction); the difference in compac-

tion between natural soil and that in embankments after tamping; and embankment subsidence over weak bedsoils (peat, silty soil).

Besides the above, as a provision for additional works and those not accounted for in the project, a contingency factor is applied to the total volume, amounting to 5-10%.

When allowing for the volume correction for the carriageway, the method of filling the shoulders must be taken into consideration. This correction is introduced with a negative sign in estimating the volume of an embankment, since the earthworks are reduced by the volume occupied by the carriageway (Fig. 144*a*).

In the case of cuttings, however, the correction for the carriageway increases the quantity of earthworks, and therefore is introduced with a positive sign (Fig. 144*b*).

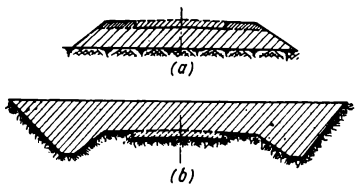


Fig. 144. Preparation of roadbed for laying pavement:
a—on embankments the shoulders are added; *b*—in cuttings a trench is dug out

The introduction of corrections for soil tamping in embankments is connected with present requirements for compaction of highway foundations, the volume weight of the soil in the roadbed often being greater than that of the soil prior to excavation. As a result of this compaction the embankment volumes are smaller, as a rule, than those of the borrow pits from which they are filled. The value of the correction factor can be determined by comparing the soil volume weight in conditions of natural occurrence with that of the soil in the roadbed.

The subsidence of embankments on sections founded on weak ground, which becomes compacted or consolidated under the embankment or is pressed out from under it, is calculated by the methods described in Sec. 44.

If the soils of separate road stretches or even within the limits of the same cross-section differ according to their workability and handling characteristics, the quantity of earthworks should be calculated separately for each soil category.

69. Length of Haul of Soil

During the construction of a road soil is excavated, transported and deposited in a different place—in a fill or a dump. Consequently, the mere establishment of embankment and cutting volumes does not constitute an adequate base for planning the organization of work and for the choice of the machinery to be used.

In conditions of broken relief requiring alternating embankments and cuttings, one of the two following methods can be employed to erect the roadbed: the soil excavated from the cuttings can be used to construct the embankment (longitudinal haul), or the embankment soil can be derived from borrow pits and the soil from the cuttings transported to spoil banks (transverse haul).

The most economic method for each individual case will be the one which will require the smallest average length of haul and will permit the most effective use of earth-moving machinery.

In many cases the means of haulage are determined by the local factors, land relief, approach roads, soil and hydrologic conditions. Thus, for instance, transverse haul is excluded on stretches with highly saline soil, on crossings over swamps, within inhabited localities and at places of agricultural importance. In the same way it is impossible to use the longitudinal haul method if the excavated soil from the cutting is unsuitable for use in embankments or if the route linking cut and fill is barred by rivers or swamps.

When compiling a project of earthwork organization for dealing with the construction of adjacent embankments and cuttings and for determining the average haul distances one can make use of the *mass-haul or soil distribution diagram.*

The cumulative curve showing the distribution of cut and fill is plotted by means of the successive algebraic summation of embankment and cutting volumes derived from the estimates of earthwork quantities. The volumes of cuttings, which are used for obtaining soil, are given positive signs, while the volumes of embankments, for the construction of which the soil may be used, are given negative signs.

The consecutive sum of the volumes is plotted along ordinates opposite the stations and intermediate points of the grade line, which serves as the x-axis (Fig. 145).

On the soil distribution diagram:

1. Any ordinate of the curve represents the algebraic cumulative sum of embankments and cuttings from the commencement of the curve to the section under consideration.

2. The difference between two ordinates ΔV is equal to the quantity of earthworks along the distance ΔL between the two cross-sections under consideration.

3. The ascending stretches of the curve correspond to cuttings and the descending ones to embankments. The maximums and minimums of the curve correspond to the points of changeover from a cutting to an embankment or vice versa.

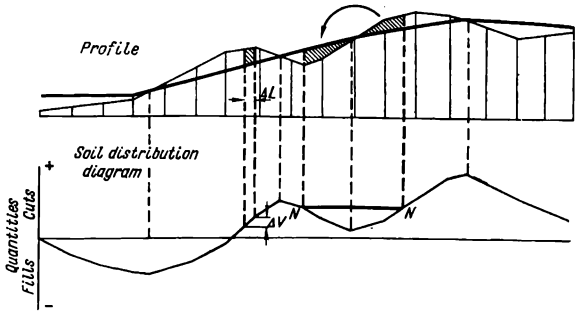


Fig. 145. Plotting of soil distribution diagram

4. The gently sloping stretches of the curve characterize small quantities of earthworks (balanced cross-sections), while the steep ones show heavily unbalanced sections requiring considerable cut or fill.

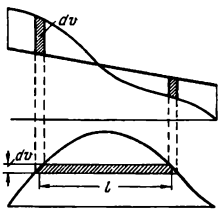


Fig. 146. Determining average length of haul

5. Any horizontal line NN, intersecting the mass curve cuts off a balanced stretch where the volume of the embankment is equal to that of the cutting. This line is called a *balance line.*

6. The average length of haul within the limits of a curve section cut by a balance line is equal to the quotient of dividing the cut-off area by its maximum height

$$L_{av} = \frac{\omega}{V} \qquad (222)$$

This feature of the curve can be proved as follows. Consider on the profile—on a stretch of a cutting—an elementary volume dv, which according to Fig. 146 is being hauled over a distance l to a fill. Assuming in a simplified way that the hauling equipment is moving along a straight line, the total haulage work required for the given volume of soil is

$$dU = dv\, l f$$

where f is the resistance to the movement of the haulage equipment.

The elementary product ldv is shown on the mass curve by a shaded strip of a height dv and a length l.

The total work necessary for haulage of the soil from the whole of the cutting into an embankment is

$$U = f \int_0^V l\,dv = f\omega$$

The integral represents the area ω of the whole cut-off part of the curve. If, however, one is to proceed from an assumed average length of haul l_{av}, then the haulage work will be

$$U = fvl_{av}$$

whence

$$l_{av} = \frac{\omega}{v} \tag{223}$$

The mass-haul diagram is of considerable value in determining the most useful earth-moving machinery in conditions of undulating terrain. Knowing the average length of haul at which each type of earth-moving equipment is used most effectively, this can be plotted on the mass curve.

The data obtained from an analysis of the mass-haul curve should be considered only as approximate and do not fully reflect the actual conditions of soil haulage. Firstly, when analyzing the mass-haul curve it is assumed that the soil is hauled along a straight line between the centres of gravity of the displaced volumes. In reality, the actual length of haul, with regard to turning of the vehicles, the possibility of moving only on stretches not steeper than specific gradients, and the siting of the approaches on an embankment, may substantially exceed this distance. Depending on the condition of the soil, the resistance to the movement may also vary greatly. Secondly, the conditions of operation of the earth-moving and hauling equipment, as well as the provision for water drainage from the cuttings during the construction period, may effectively preclude the transportation of soil from the cutting to the embankment, if the cutting is situated lower down the road grade than the embankment.

When working with power shovels it may be good practice to excavate a pioneer trench along the whole length of the cutting, beyond the economic length of haul for the particular equipment.

Substantial departures from the theoretically correct solutions reached with the aid of the mass diagram may also be occasioned by considerations of the quality of soil excavated from cuttings and borrow pits. However, more precise and reliable methods for planning earthwork organization have still to be produced.

PART V

Highway Planning and Survey

CHAPTER 11

STAGES OF THE PLANNING PROCESS

70. Types of Surveys and Their Purpose

The construction of new roads, road reconstruction and general maintenance of existing ones, of roadside buildings and other road structures are carried out according to approved projects and estimates. The construction of roads and structures is not permissible without these documents.

The road project substantiates the horizontal and vertical location of the road, the design and dimensions of the roadbed, carriageways, bridges, culverts, dwellings, business premises, etc., required for the normal operation of the road and establishes the methods to be used and the time required for the completion of the road.

The road construction estimate defines the quantity of materials required, manpower, equipment and transportation, also the cost of the separate elements as well as that of the entire road.

To prepare the data required for planning, it is necessary to carry out extensive survey work in a series of successive stages. During the first stage of the survey work the general economic aspects are studied, so as to determine the technical and economic basis of the project and its importance for the national economy. The data of the economic survey are used to draw up the plan and priority of road construction, to establish the allocations for the separate road sections and the class of the road.

Economic surveys are divided into *comprehensive* and *detailed* ones. The economic survey, which is based on the distribution of productive forces and inhabited localities, as well as on the layout of the existing road network and of other transport communica-

tions, also takes into account the anticipated development of traffic and the character of freight and passenger traffic.

Comprehensive economic surveys provide data for planning road construction and designing road networks in any given territory (district, region or republic). On the basis of the survey data it is decided what form the road network shall take, what roads are to be constructed, and in what sequence. The importance of the roads for the national economy is also determined.

Detailed economic surveys determine and substantiate the most expedient location of an individual road or bridge, the technical characteristics thereof and the general economic effectiveness of building or reconstructing it.

In detailed surveys, the total volume of freight and passenger traffic of a given road is studied in detail, taking into account long-term developments. On the basis of this analysis the most efficient direction of the road, the intermediary points and the technical features are decided.

Engineering surveys are carried out simultaneously with, or soon after, the economic ones in order to establish the horizontal and vertical road location, also the size, type and designs of highway structures and the extent and cost of work.

Engineering surveys are made utilizing the data obtained at the time of the economic investigations. The extent, character and composition of work in engineering surveys depend on the problems to be solved.

Engineering surveys are divided into *preliminary* and *detailed* ones. Preliminary surveys are the first stage of engineering surveys and they make it possible to study the road-building conditions pertaining to a given location selected with a view to the data provided by the economic survey studies. On the basis of the preliminary engineering and detailed economic surveys a decision is reached on the technical feasibility and expediency of building the road in the proposed direction. Preliminary surveys consist in studying the natural conditions in the vicinity of the road, in selecting the direction of the road and in accumulating information for a preliminary assessment of the volume and cost of the work, and the required quantity of materials, labour and mechanical equipment.

With detailed engineering surveys, the natural conditions of an area are thoroughly examined, the route marked out and data gathered to allow detailed designing of highway structures and exact assessment of the volume and cost of the work.

Before work on the road is commenced surveys are carried out to finalize the project in detail. These surveys consist in re-assessing the road location approved in the engineering project, and in acquiring data for the purpose of improving, clarifying and correct-

ing particular decisions made therein. Supplementary survey work may be executed at the time of building the road so as to improve upon the original project.

The *compilation of a road project* is an extremely complex and responsible task. Decisions adopted in the project must be technically accurate and economically sound. All of the designing and survey work must substantiate the choice of route direction in accordance with the available information on the direction and magnitude of traffic, the specified technical standards and the local natural conditions. It must also ensure stability of the roadbed, pavement and structures, the most efficient use of the capital invested in the construction of the road and the execution of the work by the specified date.

The following stages of planning are distinguished:

(1) the project report together with a summary financial estimate;

(2) the technical project and summary estimate;

(3) the working drawings.

Motor roads, individual bridge crossings and other items relating to road construction are usually designed in two stages: the project report and the working drawings. In this case it is permissible to draw up technical projects for separate complex engineering works.

In particular instances, when a road is being laid in a mountainous region or in places where there are difficult geological or hydrologic conditions, and also in the vicinity of large cities or exceptionally large or complex bridge crossings, designing is carried out in three stages, namely, the project report, the technical project, and the working drawings.

Where simple engineering items are involved, it is permissible to substantially reduce the required amount of design and estimate documents.

The number of designing stages is determined when the assignment is issued by the body that is to approve it.

The types of survey work depending on the number of designing stages are given in Table 30.

As can be seen from Table 30, in two-stage designing the project report is worked out from the data of the economic and detailed engineering surveys and also from such information as is available from former surveys.

In three-stage designing the project report is drawn up on the basis of economic and preliminary surveys, while the technical project is worked out on the basis of the approved project report and completed detailed surveys.

The technical project differs from the project report in the completeness and accuracy of the accumulated data. These data make

TABLE 30

Number of designing stages	Types of survey work	Submitted project and estimate documents
Two stages	Economic surveys. Detailed surveys. Surveys connected with working drawing stage	Project report, summary financial estimate. Working drawings
Three stages	Economic surveys. Preliminary surveys. Detailed surveys. Surveys connected with working drawing stage	Project report and summary financial estimate. Technical project, summary estimate. Working drawings, estimates drawn up from working drawings

it possible to design the road in detail both in plan and in profile, to finalize the design of the roadbed and pavement, the types of structures to be used, and to determine the precise quantities of work.

In the technical project the detailed plan of work organization and schedule of machinery and equipment operation are drawn up, and the exact quantities of requisite material, machinery, manpower, and transportation facilities needed to execute the work are established.

The estimated cost of construction work at the project report stage is determined by drawing up summary financial estimates, making use of estimates to standard and economic repeatedly employed projects. Amendments are introduced into these estimates that take into account local conditions of construction and work organization. When drawing up the technical project the cost of construction is derived from estimates compiled for the planned quantities of work involved, using standard consolidated estimates for individual structures, constructive elements and kinds of work. Where no standard consolidated estimates are available the technical project is drawn up according to individual prices and rates.

The working drawings are prepared on the basis of the approved project report or technical project. All construction work is executed in accordance with the working drawings and, therefore, these must contain the finalized details of road elements and structures.

When compiling the working drawings all the road elements (the roadbed, structures, drainage, pavement, access roads, intersections, buildings, etc.) are specified in plan and elevation. When preparing the working drawings it is not permissible to introduce modifications that lead to a lowering of the class of the road or detract from the fundamental nature of structures laid down by the project report or technical project.

The estimates for construction and erection work are compiled on the basis of the working drawings, utilizing also previous standard projects lashed to the local conditions.

71. Organization of Survey Work

Economic and engineering surveys are carried out mainly by special road designing and surveying organizations. The field work, including the preparation of the main project documents, is executed by a special survey party that collects the required materials and performs the necessary surveying work (geodetic survey, hydrometric observations, drilling operations, etc.). Upon their return, the members of the survey party, with the assistance of other employees of the department draw up the project documents. The composition of the working party may alter considerably depending on the class of road, its length, the natural conditions of the district being surveyed and the schedule which has been set for completing the work. The degree to which the district where the road is to be located has been studied previously, and the availability of materials from previous surveys, topographic maps and literature will have a very appreciable influence on the quantities of work involved and the composition of the survey party.

In Table 31 are given approximate compositions of survey parties for preliminary and detailed surveys.

The time rates for carrying out the survey and designing work are established with a view to the type of survey and the nature of the relief of the area, as well as the climatic and other local conditions influencing the efficiency of work of the party. The head of the survey party is in charge of all works and carries full responsibility for the accuracy of the survey field work, and also for the thoroughness and quality of all the data collected.

During surveys great attention should be focussed on the study of local natural conditions, and in particular on the carrying out of geological and soil investigations.

Where watercourses of considerable size are to be crossed, full data specifying the hydrologic conditions of the course should be assembled and appropriate hydrometric work carried out. Should there be talus, landslides, karsts, swamps, etc., in the route area, special investigations and geodetic operations are carried out in order to obtain exhaustive information on the conditions pertaining to designing of the road.

Among the multitude of tasks performed by the survey party considerable importance must be attached to purely geodetic functions. However, the basic task of the party is to secure all the data characterizing the natural conditions of road location sufficient

TABLE 31

Composition of party	Number of workers according to types of survey work	
	Preliminary surveys	Detailed surveys
Head of party	1	1
Assistant to head of party	—	1
Geological engineer	1	1
Road engineer	1	1
Foremen-technicians:		
chainman	—	1
leveller	—	2
catchment-area investigator	—	1
processor of field data	—	1
geologist	—	1-2
topographer	1	1
cross-section designer	—	1
Laboratory technician	—	1
Soil-sampling specialist	—	1
Driver	—	2
Store keeper	1	1
Labourers	5-8	25-30

Notes: 1. The technical staff for surveying topography and cross-section are included in the party when surveys are made in mountainous or broken country or during surveys for road reconstruction.
2. The number of geologists (engineers and technicians) may be increased depending on the extent and complexity of the geological work.

to allow the most technically correct and economically expedient solution to be reached on the project.

The composition of the survey party normally includes a geological engineer, who is in charge of all operations concerned with geological and soil investigations, and with the location of quarries for road-building materials.

Survey parties dealing with bridge crossings include a bridge design engineer.

During the course of the work the survey party may be divided into *survey groups* (detachments carrying out specific tasks under the supervision of the party head). The surveying of roads of consid-

erable length where there are many bridges, or, alternatively, where working schedules are extremely tight, may be executed by several survey parties. These form a *survey expedition.* The head of the expedition has under him a deputy who is both the chief engineer of the expedition and the chief design engineer. Included in the expedition are assistants who are in charge of geological survey operations; of the survey of bridges and other structures; of the planning of construction work organization and the provision of data for drawing up estimates; of stores, and also an accountant.

The allocation of sections of the route to the individual survey parties should take account of local conditions, with the aim of completing the survey of the entire route by the same date. Hence, the length of the route sections will be different for each party, depending on the nature of the relief, the occurrence of swamps, ravines, bridges, etc. The limits of the sections—the points where the survey parties join—are established in accordance with information obtained from cartographic data and by preliminary inspection of the territory. Usually the meeting place of two parties is fixed near an inhabited locality or a bridge.

The customary practice is for all the parties to work along the route in the same direction. If two parties are at work over one section one party usually commences at the beginning and the other at the end of the route, and they work toward each other.

Survey parties must be provided with all the necessary instruments, equipment and transport facilities.

Heads of expeditions and parties, their deputies and assistants must periodically check field work and the data as the survey proceeds. The chief engineer of an expedition will inspect the preliminary route location and its alternatives together with the head of the survey party, and then finally approve the alignment selected.

The designing and surveying departments check the quality of the survey party's work with the aid of special inspectors.

When reviewing the survey party's data the inspector concentrates on adhering to the established specifications, and checks the execution of the geodetic operations, as well as the accuracy of these operations. He also checks the completeness, accuracy and correctness of the general soil investigation and the surveys of quarries for road-building materials, and laboratory processing.

In surveys concerned with watercourse crossings the inspector checks the sufficiency, validity and completeness of all the accumulated hydrologic, hydrometric and geologic data for designing the bridge.

The cost of the survey and designing work is established on the basis of the project assignment and special tables of consolidated cost indices relating to this work.

On receipt of the assignment for carrying out the survey and drawing up the project report from the higher authority, the designing organization will proceed with the survey and designing work.

If the economic surveys have already been carried out in connection with the assignment, the initial, terminal and intermediary points through which the road has to pass will be indicated together with the approximate length and class of the road. Otherwise directions are given for making such economic surveys. Besides, the assignment will give recommendations on the types of structures to be used, a scheme of the road operation service, on where to locate the approach roads, and on the time to be allocated for completing construction of the road. At the end of the assignment the number of design stages is specified together with the date for presenting the project report.

CHAPTER 12

PRELIMINARY SURVEYS

72. Organization

Preliminary surveys of motor roads and bridges represent the first stage of an engineering survey in three-stage designing. On the basis of the information obtained during these preliminary surveys the project report is drawn up. During the execution of the preliminary surveys the fundamental design decisions are made, the most suitable road or bridge location chosen from among several alternatives, and the basic technical standards established. In the process of surveying appropriate data must be obtained for making an approximate estimate of the quantities and cost of the work and also for establishing the required road-building materials both from local and from outside sources, and whether their quantity is sufficient for construction of the road or bridge.

Preliminary surveys are undertaken on receipt of the assignment for carrying out the surveys. In this assignment the initial, terminal and the most important of the intermediary points are indicated.

The points through which the road is to pass are fixed by the economic surveys (if these have been carried out) or in conformity with the existing road network and the location of inhabited localities in areas that already have an established road network.

The season of the year does not limit the survey operations at present. With the exception of the northern regions of Siberia and the Far East, surveys in the U.S.S.R. are carried out all the year round. However, surveying operations in winter encounter considerable difficulties, which lead to an increase in cost.

The effectiveness and cost of preliminary surveys depend upon local natural conditions and the time of the year. In average conditions of relief, the normal daily coverage of a survey party is about 6-8 km. Using these figures as a guide, the head of the survey party makes out a preliminary survey progress schedule.

Preliminary surveys involve three separate periods, namely, preparatory work, field work and office processing of the collected field materials.

73. Preparatory Work

During the period of preparatory work it is essential to collect and study information obtained during former road surveys in the areas where the road will be laid. This information may be obtained

from the libraries of designing and surveying departments or from local authorities concerned with road operation. Inventory data preserved by local highway authorities will be very valuable.

If according to the assignment information and economic surveys the general direction of the route coincides with an existing road, in solving the basic question of whether to lay a new road in combination with the existing one, or to locate a road with a new alignment, it is essential to have all the information characterizing the condition of the existing road.

The climatic, general soil, geological, hydrologic and hydrogeological conditions of a locality are also studied during the preparatory period by examining all relevant literature (reference books, soil and geological maps, etc.).

When studying climatological information it is necessary to establish the general climatic conditions of the area where the road is to be laid (average monthly temperature over the year, average monthly intensity of snowfall and rainfall for a year, data for compiling a wind rose). Subsequently, during field work, all these items must be substantiated by means of information obtained from local meteorological stations and from highway authorities. The data so acquired are utilized in finalizing decisions relating to design.

If there are large bridges along the route, information should be acquired on the hydrological characteristics of the river—current velocities, discharges, maximum high water levels and gradient of the river bed. It is extremely important to acquire and study data derived from surveys, the design and building of existing bridges, control points, and gauge stations in the vicinity of the crossing. These data should make it possible to establish the best location for the river crossing, to determine the approximate span and type of the bridge, and to provide an estimate of its cost.

On the basis of the available cartographical information the location of the road is selected. The location is studied on maps of progressively increasing scale, using first those of small scale (from 1:1,000,000 to 1:200,000) and subsequently large-scale maps.

On the small-scale maps the alternative locations of the route are contemplated. In this stage questions concerning the so-called major alternatives are decided, viz., whether to lead the road around or through intermediary towns and cities, over a watershed or along a valley, whether or not to cross large watercourses, etc. These alternatives are compared according to consolidated indices with regard to construction and operating costs, as well as to the administrative and economic importance of the road.

On the small-scale map the road is traced in comparatively long straight lines; here the fine details of relief and other local factors which cannot be shown on the map are not considered.

In the location of long roads (more than 200-250 km) the curvature of the earth's surface should be allowed for and the position of the geodetic (shortest) line preliminarily established.

Having set out on the small-scale map the main and alternative locations of the road, these are studied in detail on large-scale maps, which are termed topographical, since relief (contour lines) and detailed regional situation are indicated on them.

The date when the map was surveyed should not be overlooked, as over a period of 20-30 years a district can change considerably—forests may be cut down or planted, new railways laid, canals made, new inhabited areas and industrial undertakings may appear, etc. On the large-scale maps work is commenced with the most difficult, complex and problematic areas along the proposed route, which include crossings of large watercourses, mountain ranges, swamps, ravines and the like. Having studied these areas on the map, the sections of the route which have to be surveyed in order to finalize the location of the road are established. Sections of the route passing through flat open country along extended straights may be assumed to be the main direction. The route bearings are therefore measured on the map, in order that the route may be accurately located on the site during field work. When marking out on a contour map one must refer back repeatedly to the accumulated data on general soil and hydrological factors. In broken country and mountainous areas it is sometimes necessary to extend the route, i.e., to lengthen it in order that the maximum permissible longitudinal gradients are not exceeded. When locating bridge points over large rivers it is essential to avoid crossing them near a sharp bend, a tributary confluence, or at places where islands, meanders or old river beds are situated. It is also necessary to consider how access to the bridge is to be achieved, and the nature of the geological complex in which the banks and river beds are situated. It is desirable to dispose level crossings of railways at right angles to the track and to locate them where the railway line is laid on a low embankment in open country with full visibility between rail and roadway. It is advisable to locate overpasses at places where the railway line passes through a cutting, as this will reduce the height of the overpass abutments.

Where a new route coincides with the direction of an old road, the sections of the existing road which may be usable are noted and the directions of approach roads to railway stations, docks and airports are tentatively established.

When planning roads in broken and mountainous country, a profile of the route is plotted, the grade line is planned, drainage is designed and the quantity of earthworks is determined. The final decision on the location of the road in difficult areas can only be

made after inspection of the country and completion of the geodetic survey. However, detailed and careful map work together with consideration of local factors will considerably shorten and simplify the field geodetic work.

When determining the direction of the route, the requirements and wishes of organizations and departments whose interests will be affected by the building of the road should be taken into account.

The adopted design solutions must be approved by organizations whose interests are concerned during the survey period or the design stage.

Simultaneously with map route location the main technical standards for designing the road should be established. The class of the road, the type and width of the carriageway and the overall width, maximum gradient and minimum radius of curves, etc., are included in such technical standards. Technical standards may be adjusted and finally accepted on the approval of the project report. Undue upgrading of the class of road results in excesses in designing, an unwarranted increase in construction costs and unnecessary expenditure of labour, materials and equipment. On the other hand, selecting too low a class, while reducing building costs, nevertheless lowers the operative quality of the road. The class of road and technical standards adopted must be in accordance with the potential development of traffic over a period of not less than 10 years.

The preliminary study of the survey area with the help of maps and literature will enable the probable volume of field work to be determined, also its degree of difficulty and complexity, to program the sequence of carrying out survey work, and to determine the composition and equipment of the survey parties.

74. Aerial Survey

When surveying and planning highways, especially when there is insufficient cartographical material of the region, aerial photography may be employed with advantage. Aerial photography permits the production of photographic mosaics of the country in respect to which various designing and survey work is being carried out.

An aerial survey involves a variety of interrelated air, field and office work, carried out by various means during designing of the road. The work performed involves aerial survey and processing of the results, aerial and ground investigations of the country, separate geodetic and photogrammetric measurements and surveys. Some design operations are performed on a three-dimensional model of the country.

Photogrammetric work and technical interpretation of the sequence photographs are the basic kinds of air survey work.

The main object of the photogrammetric work is to determine the form, size and the position of various local objects by measuring their images on the aerial photographs. When surveying highways by means of aerial photographs, the topography of the earth's surface is studied and the most expedient location of the road is determined. From the analysis of aerial photographs, one is able to obtain data for designing the road and for subsequent work connected with transferring the completed road project onto the ground. The study of aerial photographs makes it possible to establish the geophysical factors of a locality which must be considered when designing the road.

Photogrammetric techniques developed for surveying highways make it possible to carry out a considerable portion of the topographic, geological, general-soil and hydrologic investigations of a locality in the office. In the process of carrying out general field work information is acquired which can only be obtained by detailed inspection of individual local objects.

At present, aerial surveys are used advantageously for the selection and design of route location and bridge crossings, in geological and geomorphologic surveys, to assess water resources and for investigating vegetative cover.

Aerial photography is especially suitable where roads are to be laid in difficult mountain terrain, when, in the absence of cartographical data, routing becomes extremely difficult and demands much time and labour. Under such conditions the use of aerial photography cuts the time and cost of work. Before surveying a locality, reconnaissance work may be carried out from a helicopter so as to clear up fundamental questions such as along what river valley the route is to be laid or where a bridge crossing is to be surveyed.

When designing a road, a three-dimensional stereoscopic image of an area may be successfully employed. This is produced when two overlapping continuous photographs are examined in a stereoscope.

In the process of road design the stereomodel acts as an effective substitute for the site and makes it possible to perform a large portion of the field survey work upon it. Thus, during the period of road surveys one is able by means of the stereomodel based on the aerial photographs to familiarize oneself with the survey area, clarify and assess the geophysical peculiarities of a locality, mark out possible alternative routes and note down all natural obstacles and special features which will be encountered in locating each of the proposed alternatives.

By using an undistorted stereoscopic model of the land surface, the location of the road and structures on it can be selected, distances

and route angles measured, road stations (chainage) marked out, all basic dimensions of the road strip determined on each section of the route, photogrammetric levelling of the road carried out, etc.

In the absence of a stereomodel of the locality it will be impossible to make a special air survey of a district at the most difficult sections of the alignment, and at points where large road structures are to be situated. On the surface of the stereomodel it is possible to solve many kinds of engineering problems and produce a three-dimensional image of the most important design solutions. The use of models representing various structures which have been designed, or their various alternatives, allows a visual and objective appraisal of their advantages and disadvantages to be made and also provides a comprehensive substantiation of the solutions adopted in the project.

Design and survey work has demonstrated the great value of a helicopter for preliminary aerial surveys in highly broken country. From a helicopter it is possible to study a locality at a low altitude, moving at a low speed, and, if necessary, to hover in the air over a given point. A helicopter assists surveyors in deciphering and evaluating the quality of survey photographs.

The use of aerial surveys makes it possible to appreciably speed up and improve the quality of survey work. When selecting a bridge crossing, a photograph may show the configuration and extent of the catchment area, the width of fluvial plains, whether there are islands, rapids, shallows or semi-permanent lakes in the flood area, etc. (Fig. 147).

When surveys are to be made in swamp or forest regions one may, by means of aerial photographs, determine the limits and type of the swamp by the ecological distribution and colour of vegetation, also the approximate depth of swamps, and the nature, density and height of forests. The air survey enables the geologist to appreciate the microrelief and nature of the soil. Photographs of natural exposures permit an assessment of the density, stability and degree of weathering of rock to be made. If aerial photographs to the required scale are at hand, it is possible to avoid field instrument work during preliminary surveys. For road and bridge surveys the scale of the air survey is usually within the range of 1:10,000 to 1:30,000.

Survey experience has proved it difficult to study a locality with complicated relief and situation on aerial photographs to a scale of 1:25,000. For this reason it is desirable to have photographs at a scale of 1:5,000 to 1:10,000.

The use of a topographic stereometer, which yields photographs of the route complying with the necessary technical standards and taking full account of conditions of relief and situation, makes aerial photography even more effective for road survey purposes.

Depending on the stage of road planning, air survey operations are divided into preliminary and detailed periods, the accuracy, volume, character and schedules of work in which are established in accordance with the demands which the road must meet.

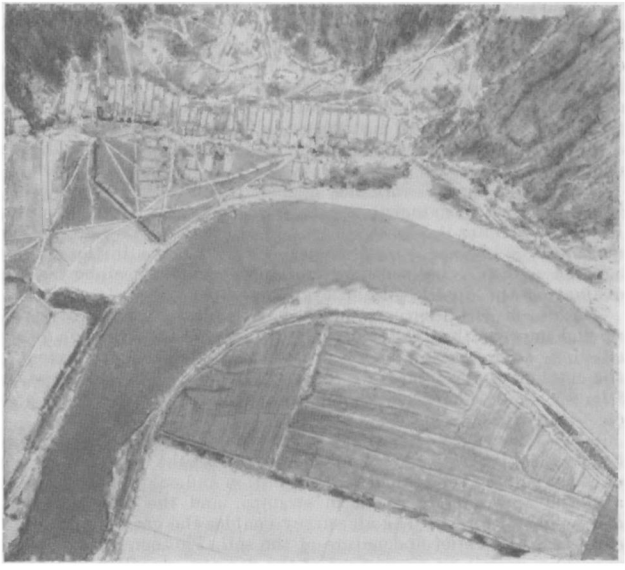


Fig. 147. Aerial photograph of a bridge crossing

The air survey should provide all necessary information and data for locating the route in the country and for compiling the design and estimating documents relating to the building of a new road, or to the reconstruction of an existing one.

Air survey methods are the most advanced and progressive among those used in road design practice. They provide for a high degree of mechanization of all the basic design and survey processes, improve their quality, rate of performance, objectivity and reliability. Thus they tend to reduce to a minimum the influence on road design-

ing of the climatic and natural conditions in the locality; they cut the time and cost of designing work and create favourable conditions for improving road survey technology.

75. Field Work

The field work in the preliminary survey consists in the precise location on the ground of the alternative routes selected on the map and in carrying out an instrumental survey in complex and difficult places. To determine the final location of the route, site investigations comprising bed soil analysis, geological and hydrological surveys are simultaneously conducted, also investigations of potential sources of road-building materials. At the same time the approximate quantities of construction work involved are estimated.

During the field survey numerous important problems arise. For their solution it is necessary to:

1. Define the initial and terminal points of the road and the conditions of route location at intermediate towns and cities.
2. Ascertain that the urban locality boundaries shown on maps conform with those on the site, taking into account their planned future development. In all cases where the proposed route will pass through or bypass large urban areas, the route location should be determined on the site by an instrumental survey.
3. When the route is laid through urban localities, determine the possibility of locating the new route along individual streets, also their plan, longitudinal profile, width, requisite drainage, the necessity of demolitions and the occurrence and location of underground services. In addition, the condition of the ground beneath the street where the new road will be laid should be investigated, as well as that of its pavement and structures.
4. Survey sections of the existing roads earmarked for incorporation into the new road.
5. Choose the sites for crossing large watercourses. Bearing in mind that the place where the bridge crossing is situated determines the general route alignment, the selected water crossings must be surveyed first.
6. Determine and coordinate the points of railway and highway intersections.
7. Determine the best route in broken and mountainous areas.
8. Collect data on deposits and existing quarries of building materials (stone, gravel, sand, etc.), data relative to the quality of the materials and methods of and equipment for their development. When required, samples of the materials are taken for laboratory analysis.

9. Examine in detail the collected climatic, soil, geological and hydrological data relating to the area of route location.

10. Collect data for drawing up the work organization plan and for financial estimates.

11. Coordinate the route location and other problems with the local authorities.

The route is laid out on site according to the map data and bearings. From the starting point of the route, usually accurately fixed (a city boundary, the junction to a major highway), the location is accurately established with a theodolite from coordinated bearings gauged on the map, allowing for the magnetic declination, or taking as a basis local features (populated areas, lakes, swamps, forests), so that the route on site will conform to that marked on the map. The route is *pegged out* on the site, the poles being spaced at definite intervals of 300-500 m on straights, whilst the bends are marked by additional wooden stakes. If the proposed route coincides with the general alignment of an existing road, then it is not necessary to peg it out along the entire length of the route, but only on those sections which deviate from the existing road. Simultaneously with route pegging out, the expedience of the route is verified and alternative routes and areas for instrumental survey are determined. Measurement of the length of the route may be simplified by the use of a range finder, pedometer or car speedometer.

In broken country several alternative routes are pegged out, which serve as a basis for selection of the final route. In extremely broken or mountainous areas, along the pegged out route a tachometrical survey is carried out over a strip 150-200 m wide. According to the contour plan the route is finalized in the office and later transferred onto the site.

A route across mountain ridges is plotted on large-scale contour maps and is finally defined during the area survey. In plotting such a route a helicopter may be of great assistance. If detailed maps are not available for locating the lowest mountain pass, barometric levelling may be used. The line is set out with a theodolite, level or gradiometer. The route should be laid out working downwards from the pass, which permits excellent observation of the terrain and leads to complete assurance that the selected route is the best one. If the slopes are very steep and if landslides may be anticipated, or no convenient approaches to the pass are available, the possibility of cutting a tunnel through the mountain should be investigated.

In areas where large watercourses have to be crossed, location plans of the crossings and river cross-sections are drawn up. Data are also collected concerning high water level, the circumstances of river freezing and ice motion, and all the information necessary for computing the opening of the bridge.

76. Soil and Geological Investigations

During the field soil and geological investigations of the area through which the road will pass it is necessary to:

1. Study the general subsoil and hydrogeological conditions of the respective alternative routes, particularly in relation to the utilization of the soil for roadbed construction.
2. Carry out geological investigations and exploration work at points of large watercourse crossings.
3. Investigate separate sections of the route where the formations are geologically complicated (landslides, screes, karsts, swamps, frost heaves, etc.).
4. Locate quarries of local materials suitable for use in road construction.

Bed soil conditions are studied mainly through examination of the available natural exposures on hill and gully slopes, and in excavations. In complicated sections test pits and trial holes are dug in order to study better separate sections along the route. In places where there is a change of contour and vegetation, and where a change in bed soil conditions may consequently be anticipated, pits 0.8 × 1.7 m in plan and 1.5-2.0 m deep are excavated. To ascertain more precisely the bed soil conditions between the pits and find the places where the soil changes, intermittent deep trial holes 0.5 m in diameter are bored. In the main trial pits soil samples are taken from different strata for field and laboratory analysis. The data on the results of route inspection and on the test pits and trial holes are entered in a special soil log-book. They are used for determining the structural classes of the soils and the possibility of utilizing them for roadbed construction.

Soil studies can be successfully carried out by means of sampling borers and hand augers. A sampling borer is a metal tube 1 m long, which is graduated at 10 centimetre intervals. This tube has a cylindrical groove in which soil collects when the rod is driven into the ground. After the sampler has been driven in, it is turned and then pulled out. From the soil retained in the groove it is possible to ascertain the disposition and thickness of the soil strata. Soil investigation with sampling borers is carried out at intervals of 50-150 m, and should a difference in soil conditions be observed, bore holes are drilled for a more precise inspection using a hand auger (Fig. 148). Hand augers are also used when digging holes for stakes. This tool consists of a duralumin scoop 20 cm in diameter, to which detachable steel blades are secured. The scoop is fixed to a tubular duralumin bar 40 mm in diameter and approximately 2 m long. Into the upper end of the bar a handle is inserted.

The general characteristics of the soil conditions along the route, and at separate geologically complex sections, are entered into an explanatory note drawn up by the geologists.

Engineering-geological surveys of bridge crossings consist in studying the geological formations of the river beds and flood plains. With this in view, trial holes are bored in the river bed and in the flood plain of sufficient depth for designing the bridge abutments. Bore holes are also drilled in the flood plain in order to investigate

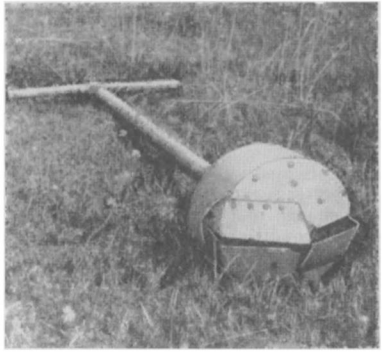


Fig. 148. Hand auger

the quality of the soil locally available for filling behind abutments and at the approaches to bridges. Borings along all selected alternatives of bridge crossings are made only to the extent necessary for obtaining a general picture of their geological structure. Usually 2 or 3 bore holes are drilled. Near the surface a large-diameter borer is used, and later it is changed to one having a smaller diameter. After all alternatives of the bridge crossing have been studied and the main location chosen, additional borings are carried out along the selected route in order to establish a detailed geological cross-section. The amount of borings necessary depends on the size of the bridge, the complexity of geological conditions, the bridge design, etc., and the work should be carried out according to Table 32.

Surveys are carried out in the field to locate and assess roadside and basic sources of construction materials. Roadside quarries, as their name implies, should be situated in the vicinity of the road. These are usually excavated during one construction season. Basic

TABLE 32

Design length of bridge, m	Geological formation at bridge crossing	Bore diameter, mm	Number of trial holes	Depth of holes, m
From 100 to 250	Simple	127/115	3-5	15-20
	Average	127/115	5-7	15-20
	Complicated	168/155	7-9	20-30

Notes: 1. If the bridge span is over 250 m, one hole is added for every 50 m.
2. If the bridge span is 10-30 m, two holes 10-12 m deep are bored, as a rule, one hole at each extreme abutment.
3. The depth of the bore hole should extend below the depth of the abutment foundation at least 3 to 5 m, depending on the complexity of the geological formation.

quarries may be located at an appreciable distance from the route and can be used to supply various high-grade construction materials for the entire road, or for a considerable part of it. The materials are mainly delivered to the site by motor vehicles, or by rail, or waterway. Field surveys consist in verifying the data collected on the existing quarries during the preliminary period, and in prospecting for new deposits by surveying the area in the vicinity of the route.

The width of the strip surveyed when searching for sand does not usually exceed 20 km, and when searching for stone or gravel—40 km (20 km on each side of the route). If within the limits of this strip no deposits are found, then larger quarries and deposits, situated at a greater distance and containing materials quite suitable for road construction, must be investigated.

During the field work a log-book is kept, where the itinerary and the location of bore holes and of natural and artificial exposures are entered. A sketch map showing the location of the deposit is entered in the log-book together with the deposit area, capacity, the stripping soil category, material formation, assumed reserve and suitable methods for quarry development and material transportation. The quality of material from deposits in the vicinity of the route is assessed initially by visual inspection and by selecting samples for testing in laboratories. To evaluate the granulometric composition and filtration coefficient of sand 2 or 3 samples should be taken. To determine the granulometric composition and the field petrographical components of gravel it is usually sufficient to take 1 or 2 samples.

The quality of stone is determined visually. Samples taken for laboratory testing are usually 5 × 5 × 8 cm in size.

When investigating the soil it is also advisable to determine whether it is possible to use for road construction purposes the various residues from local industries (slag, broken brick, rubble, etc.).

For working out the project report a preliminary plan for supplying the road with the main construction materials is drawn up, showing the average length of haul and the zones to be supplied from each quarry.

77. Field Processing of Survey Data

The processing and registration of the survey data in good time is of great importance. In field conditions it is essential to review daily all the work that has been carried out, make the necessary calculations, compile the field logs, and execute the required drawings (plans, profiles, geological sections, sketches of existing bridges, etc.). As a rule, at the time of processing the data all oversights and shortcomings come to light and can easily be rectified on the following day while the survey party is still in the same area. On the basis of the data provided by the gradiometric and levelling log-books, the plan of the route at various sections and the profile are drawn. Over the sections where instrument work has not been carried out, the plan of the main and alternative locations is plotted on a map to a scale of 1:200,000-1:500,000, which is detailed in accordance with the survey data. The profile is drawn according to the topographic map and the levelling data obtained on separate sections.

If along the route there are large watercourses, a detailed plan of the river is drawn, showing all the alternative bridge crossings, as well as the existing bridges. The section of the river in the vicinity of the bridge crossing is shown on a plan in contour lines. In addition, a profile of the crossing according to the selected alternative is plotted.

Office processing of the preliminary survey data is the first stage in drawing up the project report. The most important design solutions are adopted and substantiated during the period of survey work in field conditions.

The following data are usually compiled by the survey party:

(a) a topographic map to the largest possible scale (not less than 1:500,000) on which the alternative routes and the location of quarries are indicated;

(b) a profile of the more complex sections of the route;

(c) a route plan with alternatives at a scale of 1:20,000;

(d) plans of difficult and complex places—swamps, ravines, watercourse crossings, sections of route development, etc.;

(e) cross-sections of the road where it passes along an existing road, a hillside and in urban localities;

(f) field prospecting and deposit location log-books;

(g) field records of the search for construction material deposits;

(h) laboratory analyses and test data;

(i) chart showing length of haul of materials;

(j) an explanatory note giving the characteristics of all deposits;

(k) a register of soil investigation and of existing road and structure inspection;

(l) a calculation of bridge openings over large watercourses;

(m) documents relating to the approvement of design solutions by organizations which they concern;

(n) all original field log-books;

(o) field explanatory notes substantiating the selected route location and the adopted design solutions. The explanatory notes are illustrated with photographs, diagrams and sketches.

CHAPTER 13

PROJECT REPORT

The project report for the building or reconstruction of a motor road is drawn up with a view to determining the technical practicability and economic expediency of building a road along a given direction. The project report substantiates the approved technical standards, route direction, contemplated quantities, methods and costs of work.

The quantity of detailed material and the composition of the project report when designing the road in three or in two stages vary, since in three-stage designing the project report is compiled from preliminary survey data, and in two-stage designing, from detailed surveys. In the latter instance, therefore, the project report must be worked out to a higher degree of accuracy and in more detail.

In drawing up the project report for building a bridge, the site selected for the crossing, the bridge opening and span, the proposed engineering standards, and the bridge design selected are substantiated. In addition, the quantities of the main construction work, the required materials, equipment and manpower, and the methods to be employed for carrying out the work are tentatively stipulated.

All subsequent survey and planning work is carried out in accordance with the project report.

78. Selection of Engineering Standards

Engineering standards for road design are substantiated and selected on the basis of the assignment data for carrying out the survey or on the economic survey information. In the process of preliminary surveys engineering standards may be more accurately established.

The main engineering standards established in the project report include the class of road, the overall width of the road and the carriageway, the maximum longitudinal gradient, the minimum radius of curvature, the type of carriageway, the maximum design load and the dimensions of structures.

Using the economic survey data, a detailed explanatory note and a graph showing the distribution of traffic and its intensity along particular sections of the road are compiled. The technical class of the road and the engineering standards are determined according

to the anticipated annual average daily flow (A.D.F.) in both directions, depending on the relief of a locality.

Since the traffic intensity and the relief of a locality may vary for different sections of the road, on roads of a considerable length separate sections may be of different technical classes.

All engineering standards are established in relation to the *traffic requirements*: i.e., design speed, traffic intensity, composition and character of traffic. The relation of engineering standards to traffic is briefly as follows:

The *design speed* of traffic affects the determination of the traffic lane width, the minimum radius of curvature, the maximum longitudinal gradient, the minimum sight line in plan and profile, the minimum radius of vertical curves and the type of carriageway pavement.

The *nature* and *composition* of traffic are taken into account when specifying the design and thickness of road pavements, the type and the design of structures, and the maximum longitudinal gradient.

All geometric road elements must be so designed as to offer the minimum impedance to the flow of traffic. Hence, in designing the tendency should be to specify radii larger than the permitted minimum ones for horizontal and vertical curves, and shallow longitudinal gradients, provided this does not cause a considerable rise in construction costs. In Chapter 9 the recommended standards for roads of classes I-III are stated: their application should materially assist in elevating traffic speeds.

Standards for designing the roadbed, drainage, openings of minor structures, etc., are determined in relation to local climatic, general soil, hydrogeological and hydrologic conditions pertaining to the specific sections of the route.

The basic question demanding careful study of local conditions is the choice of that type and design of road pavement which will permit the maximum use of local road-building materials in its construction.

The total building cost is considerably affected by correct specification of the type of pavement. When designing a road on which a considerable growth of vehicular traffic intensity is expected (10 years) over its level at the time of survey, a *programmed* (step-by-step) laying of the road pavement is permissible.

Programmed construction of a road may be planned in one of the following ways:

1. The roadbed horizontal and vertical location is designed to accommodate the final predicted traffic flow. The initial number and width of traffic lanes, however, are determined by the existing flow and are designed with a view to gradual future expansion as the traffic flow develops. Thus, it is possible to plan for an initial

carriageway width of 3.5 m and allow for its subsequent increase to the required full width of 7 m at a later stage.

2. The road pavement is initially constructed to the full planned width, but it is improved gradually. For example, during the initial construction of the road a gravel or rubble pavement may be specified. Later, when the traffic intensity has grown, the thickness of the pavement may be increased or the road stabilized with a bitumen or tar binder.

3. The road pavement is improved gradually; simultaneously the carriageway is widened and the pavement structure and surfacing modernized.

It is essential that the method used for the programmed construction of a road pavement be worked out in accordance with the anticipated traffic growth, so that at any period the condition and width of the surfacing will suit the traffic requirements.

The programmed method of road building is being more and more developed, since it reduces the initial capital investment for road construction and spreads the capital investment over a longer period of time, which leads to an increase in the number of roads being constructed at a given time. The schedules for the programmed construction or reconstruction of a road are worked out on the basis of economic surveys relative to fluctuations in traffic intensity for the anticipated design period and on the anticipated wearing qualities of the road pavement selected.

If a road of considerable length passes through different climatic zones, it is essential to vary the engineering specifications to suit the particular district through which the road passes.

When adopting engineering standards it is necessary to take into account the importance of the road being designed for the economic and cultural life of the territory served.

79. Estimate of Work Quantities and Cost

When drawing up the project report, the quantities of material and work involved must be estimated as accurately as possible and their approximate cost established. It is particularly important to determine the exact quantities of basic work required for the construction of the roadbed, for laying the road pavement and for building the structures, which are responsible for about 70-75% of the total cost.

Table 33 gives an approximate distribution of the estimated costs for the individual elements of road construction work under separate headings and by kinds of work.

In three-stage designing it is possible to determine only the quantities and cost of the main kinds of work, taking the cost for the

TABLE 33

Headings and kinds of work	Percentage of the total estimate of road cost	
	Flat country	Mountainous country
Development of route and preparatory work	0.5	2.0
Roadbed	10.0	35.0
Structures	12.0	25.0
Road pavement	53.0	15.0
Roadside buildings and their equipment	7.0	1.5
Road furniture and accessories	1.6	1.1
Miscellaneous	1.5	6.0
Supervision	0.2	0.2
Temporary structures and acquisition of equipment	11.0	11.0
Other expenses	0.8	0.8
Contingencies	2.4	2.4
Total	100.0	100.0

remainder of the work from project figures for other roads built under similar circumstances. If along the route there are complex engineering structures (retaining walls, bridges, etc.) in assessing the quantities of work the existing designs of such structures may be utilized and adapted to suit local conditions.

The approximate quantities of earthworks are determined in the project report according to:

(1) the data of technical projects for the construction of roads designed in similar conditions and according to the same engineering standards;

(2) the elevation differences and the design of the roadbed established for separate characteristic sections of the road;

(3) tables (on sections where instrument surveys were carried out and the profiles plotted);

(4) characteristic cross-sections plotted for parts of the route having considerable transverse gradients in broken and mountainous country.

When assessing the quantities of work, the soil structural group and methods of work are determined simultaneously from the general soil investigation.

The quantity of work involved in strengthening embankment and cutting slopes, the bottom and sides of ditches, and in the provision of velocity-reducing steps, channels and chutes may be determined from relevant data obtained from previous road construction under similar conditions. If there are no such data available, the stabilization of slopes is designed for separate characteristic sections, and the resulting data are adopted for the entire length of the road.

The sizes and number of minor structures are established largely in the process of field survey work. In two-stage designing, openings are determined on the basis of hydrologic calculations, and taking into account the dimensions of existing structures. It is also necessary to provide for a certain number of overflow structures whose dimensions have not been computed.

The total number of structures, their dimensions, material and design are adjusted in the light of road building practice acquired under similar conditions.

When choosing the type of structure, the availability of local building materials from which these structures may be built should be taken into account. Thus, for roads of inferior classes running through forest areas, wooden structures may be specified; in areas abundant in stone, rubble drains, bridges and filter beds are widely used. For highways of the highest classes road structures are, in the main, designed of reinforced concrete, and, occasionally, steel.

The type and design of the carriageway are worked out for separate characteristic sections of the road in relation to the available quarries and also to the general soil and hydrologic conditions of the area through which the route passes.

When selecting the type of road pavement, several alternatives of road pavement are compared according to cost of work, service life and convenience of carrying out the construction work.

The main portion of the cost of road construction consists of the cost of local and imported materials, which, in turn, is determined by the magnitude of transport costs and depends on the length of haul. The correct average haulage distance is established with a view to the distribution of quarries in the area of the route and their capacity, i.e., to the quantity of usable materials.

The limits of the supply zones from each quarry and the average haulage distance are usually determined graphoanalytically.

Let us assume that in the route area three suitable quarries K_1, K_2 and K_3 are located, as indicated in the road plan (Fig. 149). Further, imported stone material may be obtained from the rail-

way station at the origin of the road. The local quarries are situated at distances of l_1, l_2 and l_3 from the route, respectively.

If the cost of 1 m³ of material in each quarry is A_1, A_2 and A_3, respectively, and the transportation cost of 1 m³ of material from each quarry to route location is B_1, B_2 and B_3, the full cost of 1 m³

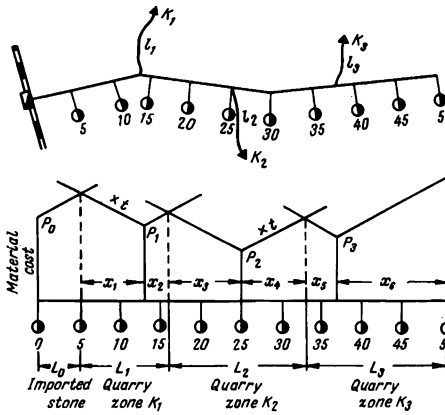


Fig. 149. Chart for determining material supply zones and haulage distances from quarries

of material at the point of emergence of the haulage road from the quarry to the route will for each quarry be equal to:

$$P_1 = A_1 + B_1; \qquad P_2 = A_2 + B_2; \qquad P_3 = A_3 + B_3$$

Let us denote by P_0 the cost of material delivered at rail sidings, i.e., the cost of imported stone at the railway station.

To determine the building material supply zones, a *cost chart* is plotted. The continuous chainage of the road centre line—with no indication of relief or situation—serves as the x-axis on this chart, which is plotted to a scale of 1:50,000-1:20,000. The length of sections with different types of pavement is shown along the x-axis.

At the junctions of the approach roads from the quarries with the route, perpendiculars are drawn on which the height corresponding to the cost of quarry material P_1, P_2, etc., is plotted to some convenient scale. These perpendiculars serve simultaneously as ordinates. In Fig. 149 costs P_0, P_1, P_2 and P_3 are shown. Let us

denote the cost of transporting 1 m³ of material along the route over a distance of 1 km by the letter t. Then the cost of the material at a point situated at a distance x from the junction of the approach road will be equal to $P + xt$.

It is known that the expression $y = P + tx$ is the equation of a straight line intersecting the y-axis at the ordinate P, and inclined to the x-axis at an angle whose tangent is t.

The plotted chart shows the variation in cost of 1 m³ of stone from every quarry at any point on the route. Intersection of the straight lines of unit cost for adjacent quarries indicates that at a given point the material from both quarries costs the same and, therefore, this point is the boundary of the supply zone for stone brought from each of the quarries.

In Fig. 149 the intersections of the cost lines for quarry materials are shown, as well as the supply zone limits for each quarry. The length of the supply zone from quarry K_1 is indicated by L_1, from quarry K_2 by L_2, from quarry K_3 by L_3 and the supply zone for stone brought from the railway siding by L_0. The stone consumption in cubic metres per 1 km of road will be designated by q. The gross requirements for material for each of the four sections will be expressed in the following way: on the first section by $Q_0 = qL_0$, on the second by $Q_1 = qL_1$, on the third by $Q_2 = qL_2$ and on the fourth by $Q_3 = qL_3$.

These figures must be balanced against the known material reserves of each quarry. Where it is found that the reserves are insufficient the necessary amendments should be made to the supply zone limits indicated.

The average cost of stone for each of the sections can be estimated by graphoanalytical method as the height of a rectangle having an area equivalent to that of the respective cost chart area on the section under consideration.

The average weighted haulage distance from quarries is determined according to the haulage "moments" from each quarry as set out in Table 34.

Thus, the average haulage distance from the three quarries will be

$$l = \frac{\sum_{1}^{6} M}{Q} \text{ km}$$

Calculations of this type are made for every sort of material required—stone, gravel, sand, etc. These figures are used for drawing up the financial estimate. For greater clarity a haul diagram for construction materials is plotted on which all quarries, supply zones, haulage distances and the quantity of material used from each

TABLE 34

Section No.	Length of section, km	Quantity of materials required per section, m³	Haulage distance from quarry to route location, km	Average distance of haulage within a section, km	Haulage moment M on section, m³/km	Remarks
1	x_1	x_1q_1	l_1	$l_1+\frac{x_1}{2}$	$M_1=x_1q_1\left(l_1+\frac{x_1}{2}\right)$	$x_1+x_2=L_1$
2	x_2	x_2q_2	l_1	$l_1+\frac{x_2}{2}$	$M_2=x_2q_2\left(l_1+\frac{x_2}{2}\right)$	
3	x_3	x_3q_3	l_2	$l_2+\frac{x_3}{2}$	$M_3=x_3q_3\left(l_2+\frac{x_3}{2}\right)$	$x_3+x_4=L_2$
4	x_4	x_4q_4	l_2	$l_2+\frac{x_4}{2}$	$M_4=x_4q_4\left(l_2+\frac{x_4}{2}\right)$	
5	x_5	x_5q_5	l_3	$l_3+\frac{x_5}{2}$	$M_5=x_5q_5\left(l_3+\frac{x_5}{2}\right)$	$x_5+x_6=L_3$
6	x_6	x_6q_6	l_3	$l_3+\frac{x_6}{2}$	$M_6=x_6q_6\left(l_3+\frac{x_6}{2}\right)$	
Total	$\sum_1^6 xq=q\,(L_1+L_2+L_3)=Q$				$\sum_1^6 M$	

quarry is shown. All the materials to be delivered to asphalt concrete and cement concrete factories (bitumen, powdered minerals, rubble, sand, cement and gravel) and materials which are conveyed from quarries to the route location are considered separately. Thus, for example, the average haulage distance and, consequently, the cost of broken stone will be different when it is conveyed to the roadside for placement into the roadbed, and when it is transported to the factory for the preparation of asphalt concrete mixes.

The number, dimensions and types of minor structures are determined after calculations of their openings, making maximum use of standard projects. The design discharge is calculated according to the relevant formula, while clearances and loads are taken in accordance with the assignment and the road class.

The number and types of roadside buildings in the project report are established depending on the length and class of road, on the number of inhabited localities situated along the route, and on the possibility of making use of existing buildings. The cost of construction of roadside buildings is estimated: i.e., equipment servicing

stations, maintenance workshops, houses for road and bridge engineers, vehicle servicing stations, etc.

In addition, the necessity of constructing temporary buildings to house the administrative and technical personnel and workers during the road construction period must be taken into account.

The quantities and cost of work for special structures such as retaining and supporting walls, drains, galleries, overpasses, tunnels, etc., are calculated on the basis of standard or special projects with a view to the experience gained during the building of structures of the same type under similar conditions.

The cost of construction in the project report is determined by a financial computation based on estimates for standard economic projects with the introduction of correction factors to allow for local construction conditions.

80. Work Organization Plan

The work organization plan at the project report stage is developed with varying degrees of detail depending on the number of designing stages adopted. In this section the work organization plan based on the preliminary survey for three-stage designing is considered.

In drawing up the work organization plan the methods for completion of the work within the allotted time are established on the basis of the estimated work quantities, at the same time providing for high-quality and economical performance of the work.

The work organization plan defines the methods for carrying out the work at individual sections, the target dates for completion of specific items of work; the number and types of tools, plant, machinery and equipment and the duration of their work; the location of asphalt and cement concrete mixing plants; schedules of machine utilization and transportation during road construction work (diagrammatic); the transportation requirements for the supply of construction materials; the total requirements for stone, gravel, sand, bitumen, wood, metal, cement, etc.; the requirements for manpower and temporary dwellings, service and production premises.

The documents pertaining to work organization include an explanatory note, a summary list of work quantities, a schematic general construction plan, a work performance schedule, summary lists of the overall requirements for manpower, machines, transportation facilities and equipment, and a summary list of road construction materials.

The work organization plan is presented in the form of a road construction progress chart giving the target dates for completion of the basic work (Fig. 150).

When compiling a progress chart it is desirable to avoid seasonal peaks in the construction work, and attempt to distribute it throughout the whole year as far as practicable. However, good practice dictates performing some kinds of work in the summer.

81. Content of Project Report

The content of the project report depends on the number of design stages adopted. With two-stage designing the volume and content of the project report is fuller, and it will include additional material compared with a report in three-stage designing.

The project report submitted for approval to the higher authorities consists of several parts, each covering separate problems, and each part, in turn, will include different materials, viz., explanatory notes, maps, schemes, diagrams and lists.

The project report consists of the following parts:

Part I. General Notes.
Part II. Technical and Economic Substantiation.
Part III. Technical and Economic Indices.
Part IV. Roadbed and Pavement.
Part V. Structures.
Part VI. Buildings and Structures of Operation Department.
Part VII. Auxiliary Structures.
Part VIII. Construction Materials.
Part IX. Construction Work Organization.
Part X. Financial Estimates.

The general notes give a concise description of all the parts of the project report, the basic technical solutions and the technical and economic characteristics.

Parts II and III characterize the area of road location and give data relevant to the part the road is intended to play in the national economy, the expected traffic flow, justification of the road class adopted and of the adopted engineering standards and design. Here also descriptions of alternative routes are given, their comparisons and the reasons for the final selection of the chosen route.

Part IV of the project report includes profiles of separate difficult sections of the road, cross-sections of the road including the road pavement, and a preliminary list of the earthworks necessary for construction of the road.

In respect to structures a general outline of local geological conditions, lists of minor structures and medium and large bridges and overpasses are given. For large bridges possible alternative bridge crossings are described and compared, together with relevant hydrologic calculations and the alternative designs from which the submitted bridge was chosen.

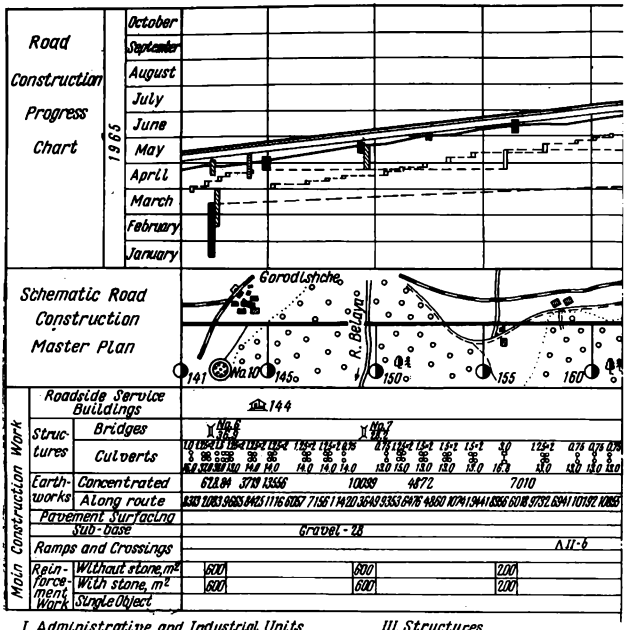
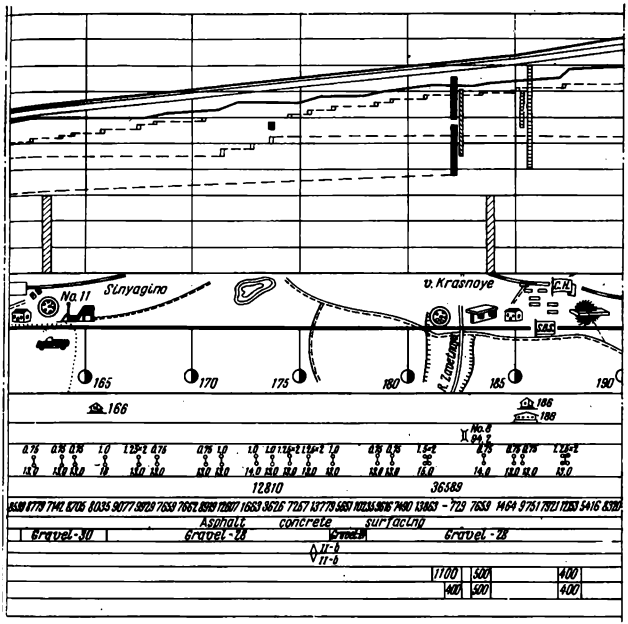


Fig. 150. Road con



5. Bridges and viaducts

6. Permanent operative service buildings

7. Temporary buildings

8. Concentrated earthworks

△ VI. Access roads at grade

Name of Ministry or Department		Name of Design Organization	
Design Assignment		Road-building Organization	
Highway X-Y			
Ch. Eng. of Hw. Des. Inst.	Signatures	Name	Road Construction Progress Chart
Design Chief Engineer			
Head of Department			
Chief Expert			
Head of Group			Scale 1:100,000
Checked			Drawing No. on sheets
Computed			Date of issue

struction progress chart

In part VI the reasons for accepting the selected operating department organization and the arrangement of roadside buildings along the route are substantiated.

Road intersections at the same or separate grades, slip roads and railway crossings, road signs and measures for the prevention of snow drifts are described in part VII of the project report.

In part VIII the characteristics of the local building materials are given in detail, also the description and type of quarries and materials, excavation conditions and construction material transportation facilities.

In part IX reasons for the selection of the particular work organization plan are given as well as the quantities of the main work, a tentative road construction plan and a brief description of the proposed methods of work execution. The yearly requirements for building materials, prefabricated constructions, transportation, machines, electric power and manpower are given. The quantity, capacity and arrangement of production enterprises are substantiated, and also the requirements for temporary buildings and structures.

Technical and economic indices are given to characterize the organization of construction.

A financial estimate of the cost of execution is compiled in the project report on the basis of consolidated estimate indices.

All the materials of the project report are filed in separate volumes by parts.

All drawings must be produced on standard size sheets, viz., 29 × 20 cm, drawings larger than the standard sheet must be folded to the standard size. The route plan is to be drawn to a scale of 1:10,000 with North at the top of the sheet, as far as possible. If, however, the route runs from South to North, the sheet is oriented with North at the right-hand edge. The profile is drawn as shown in Fig. 9.

The explanatory note must be brief, clear and precise, and include photographs and diagrams of individual characteristic and difficult sections of the route, and of bridge crossings. The explanatory note must substantiate the basic technical solutions which have been taken, and the technical and economic indices of road construction.

The completed project report is submitted for approval, and correction, if necessary. If the road is being designed in two stages then after approval of the project report construction work may be commenced. However, with three-stage designing the project report serves as the initial material for the subsequent detailed survey and the compilation of the technical project of road construction.

CHAPTER 14

DETAILED ENGINEERING SURVEYS

82. Survey Procedure

Detailed engineering surveys are carried out, as a rule, for compilation of the project report and, less frequently, of the technical project.

The survey party which performs the detailed road or bridge crossing survey carries out all the instrument work, survey measurements and investigations necessary on the site in order to establish the route location in plan and in profile, to design the road pavement and structures, and to compile the estimate sheet.

The composition of the survey party and the instruments and equipment which it is outfitted with must correspond to the expected conditions of work. Special care must be taken in outfitting parties for surveying sparsely populated and remote districts.

During the preparations for the party's departure schedules of the survey work, the necessary documents, materials and stocks, field log-books, forms, calculation tables, etc., are prepared.

The assignment (instructions) to carry out the detailed survey is given to the head of the party. Simultaneously, relevant available cartographic material, information on existing bench marks and the magnetic declination, on the climatic characteristics of the area to be surveyed, on river crossings and types of existing bridges is gathered. Before departure all the geodetic instruments and hydrometrical implements must be checked.

The progress of the survey party depends on the general topography of the area (flat land, broken or mountainous country), also on the density of population, the bogginess of the land, etc., and on the number of inhabited localities. In flat country the daily progress of the survey work may average 5-5.5 km, in mountainous regions the progress is much slower, only 0.5-1.5 km. It is possible to speed up survey work by using more efficient methods, viz., helicopters for surveying the route, aerial photography, geophysical methods of soil exploration, automatic boring implements, etc.

During the detailed survey the survey party transfers the road location to the ground and pegs it out. The party then carries out levelling of cross-sections and profiles; maps out the difficult areas in contour lines; collects data for calculating and designing minor structures and drainage installations; makes morphometrical and, if necessary, hydrometrical measurements at crossings over large

rivers; performs geological engineering soil investigations in the vicinity of the route; carries out preliminary investigations of road-building material sources; permanently stakes out the route on the land, and processes the gathered materials.

Where the route follows an existing road, the conditions of the earth foundation, carriageway and road structures are studied.

Survey work is conducted in a specific order, the survey party being divided into separate *groups*, or *teams*, carrying out the following complete operations:

(1) *The party head's group* transfers the location of the road to the ground and cuts through forests, ranging the line and establishing the position of the turning angles.

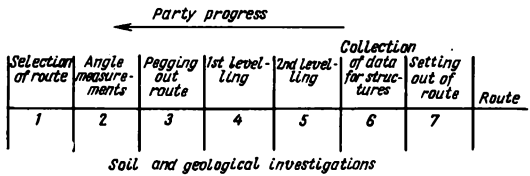


Fig. 151. Survey party progress chart and division into groups (1-7)

(2) *The group of the party head's assistant* measures the turning angles, selecting the curve elements and lashes the route to trigonometric points.

(3) *The stationing group* measures the route, surveys the features of the adjacent terrain, marks out cross-sections, makes sketches and diagrams of existing structures and keeps the chainage log-book.

(4) *The 1st leveller's group* surveys the levels of all the main points along the main and alternative routes, and sets out bench marks.

(5) *The 2nd leveller's group* performs control levelling between the major survey points, surveys cross-sections and catchment area gradients.

(6) *The group specializing in minor structures* surveys the catchment areas, collects data for hydrologic calculations, compiles diagrams of existing structures and studies their serviceability.

(7) *The group for finalizing the layout* permanently fixes the initial, terminal and intermediate aligning points and the turning angles.

The work of the survey party comprises a series of interrelated operations, carried out consecutively by the groups (Fig. 151). The

successful work of each of these groups is determined by the working area limits prepared by the preceding group. For carrying out certain tasks (for example, the mapping of a locality) the groups may be united.

The geological engineer, soil expert (if required) and drilling foreman make up a separate group, which conducts its work under the guidance of the party's head concurrently with other groups.

83. Route Selection

Taking as a guide the general location of the route marked on a map or selected during preliminary surveys, the head of the party investigates the topography on a section which will provide work for the party for several days. Upon inspection and survey of the land the route control points are established and at these places reference points are noted, such as trees or buildings, or, alternatively, special large poles are set up. When laying a road in unfavourable soil and geological conditions the geologist and soil expert are called in to carry out an inspection of the locality. In difficult topographic conditions (broken country, swamp land, ravines), where the road location can be determined only after processing of the materials obtained from field surveys, the points where the main and tentative routes are to be located are marked out.

Surveys are commenced by ranging the route line in the selected direction. A theodolite is set up on the centre line of the road at its origin for ranging the line. According to geodetic rules the line is ranged *toward the theodolite*. On long straight stretches intermediate stations are chosen for the instrument. The ranging poles may also be set up using binoculars having a magnification of 6 or 8 times. The poles are positioned at regular intervals of 50-100 m depending on the relief, so that the men measuring the route with a tape-line (chain) should see at least 2 poles in front of them. The location over hills, ravines and other obstacles is ranged according to methods described in the courses on geodesy and in special instructions. When ranging a line using magnetic compass bearings it is essential to allow for the magnitude of the magnetic deviation.

When laying out a road through a forest, the brushwood should be grubbed out to a maximum width of 1 m. In all cases care should be taken to keep the number of trees felled at a minimum, removing only the branches which obstruct surveying work. For the main and tentative routes existing cuttings and roads are utilized where practicable. As the survey party progresses, the ranging poles are removed and "substitutes" put in, which are solid stakes 1.5 to 2 m in length. Afterwards, when the alignment is fixed, permanent poles are put in at a distance of mutual visibility and centre

line posts are driven in at an average distance apart of 2 km, and also at difficult points along the route (at bridge crossings, road intersections, inhabited localities and the like).

The selection of the correct position for the apex of a horizontal curve and of the curve radius is of very great importance. The minimum number of bends is established at points where obstacles are bypassed, i.e., when cutting through broken or mountainous country, at normal crossings of watercourses, railway lines and motor roads, and along urban streets.

The largest possible curve radius must be selected (within 2,000-5,000 m) which will ensure a high traffic speed without requiring the application of super-elevation, transition curves or widening of the carriageway. The selection of a smaller radius is permissible where it can be justified by economic factors, as, for example, in mountainous country or urban areas.

When determining the curvature of a horizontal bend one should be guided by the following considerations.

1. When bypassing an obstacle the road centre line must be situated at such a distance from the obstacle as to ensure the normal layout of the roadbed and structures, allowing a certain excess of working margin. The selected radius is immediately checked on the site by comparing the curve bisector and the distance to the obstacle.

2. When crossing watercourses, railway lines and motor roads the road curvature is selected so as to provide the required straight insert between the tangent point and the beginning of the bridge, overpass or crossing, the length of the insert being not less than the length of a transition curve.

3. When designing a route it is possible that two curves in the same direction will be adjacent to each other. The radii of these contiguous curves are frequently selected so that they have a common tangent point, the end of one curve being the beginning of the other one without a straight insert (Fig. 152). The radii of these curves may differ. If the curves are situated close to each other, it is possible, in certain cases, to replace the two angles α_1 and α_2 with one angle $\alpha = \alpha_1 + \alpha_2$, designing a single curve having a radius R_3. This method may be adopted also for solving the reverse problem, when one large curve angle is divided into two. This facilitates the transfer of the curve onto the ground.

4. When laying out roads with two contiguous curves in opposite directions, so-called *reverse curves* (Fig. 153), the radii of these curves are so selected as to leave between them the required space for a straight insert. The insert must be sufficient for location of the transition to the required super-elevation, of the transition curves and of the additional width needed for each curve. If the length of the

tangent of one curve is T_1 and of the other T_2, while the lengths of the transition to super-elevation, or of the portion of the transition curve situated along the straight, are respectively t_1 and t_2, then the minimum distance between the apices of the reverse curves will be

$$L = T_1 + T_2 + t_1 + t_2$$

5. Structures should be situated on straight stretches of the road, although they may be arranged on any combination of horizontal

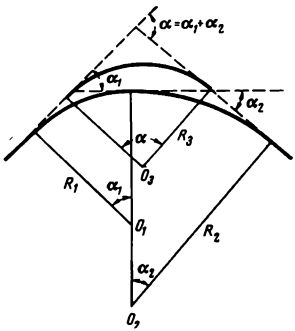


Fig. 152. Setting out curves on two adjacent bends in the same direction

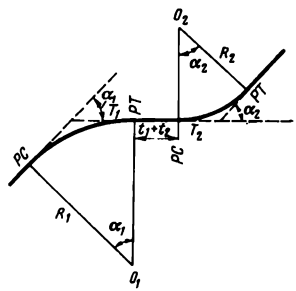


Fig. 153. Reverse curves

and vertical road elements. When laying out structures on curves the radius is so selected that not only the structure, but also the accesses to it are located on the same curve. Positioning of only a portion of a bridge on a curve complicates its construction.

6. When laying a road in urban localities or in country where deep cutting is necessary the requirements of visibility must be borne in mind. The choice of a sharp curvature may cause considerable extra expenditure in order to provide visibility (trimming away slopes, demolishing buildings), at the same time a small radius will create additional obstruction to traffic. In these circumstances it is necessary to consider the possibility of selecting a larger curve radius, since sometimes an increase in the main quantities of work is compensated by a reduction in the quantities of supplementary work required for providing adequate visibility. In all cases it is desirable that the radius selected should be justified on technical and economic grounds.

In the process of selecting the location, instances may occur when the best decision is not immediately apparent and the route is subsequently finalized by the comparison of alternatives. In such cases the head of the party marks out on the site the direction of all the alternatives, along which instrument work is then carried out. Instances are also possible when the adopted route proves to be unsuccessful owing to insufficient investigations of local conditions (landslides, swamps, saline soil). In such instances the selected location must be abandoned and a new route surveyed.

84. Measurement of Angles

Following the party head's group and in close liaison with it comes the group of the party head's assistant, comprising a qualified engineer and 3-4 workers. Liaison between the head of the party and this group is essential because in some cases the route alignment is ranged with the aid of a theodolite, while in a number of cases the instrument is required for selecting the direction of the route. With small survey parties selection of the route, ranging of the line and measuring the angles is done by a single group supervised by the party head.

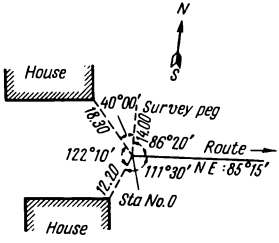


Fig. 154. Setting out the route origin

Theodolite work is commenced by fixing on the site the local terminal point of the survey route. For this purpose the theodolite is set up and accurately centred over station No. 0 and the horizontal angles are measured, while the distances to the nearest permanent objects (corners of buildings, structures, telegraph poles, etc.) or to specially driven pegs are measured with a tape line. When fixing the initial or terminal point it should be lashed to at least three permanent points or bench marks (Fig. 154). The pegs or posts used in finally establishing the route are 1.5-2 m long and have a diameter of 15-20 cm (Fig. 155). A cross-piece made from blocks of wood 60 cm in size is fixed to the lower part of the post. The post is driven in to a depth of 1.0-1.5 m; such a post is sometimes used as a temporary bench mark.

The theodolite station at the origin is also used for ranging the alignment. If the apex of the first angle is located within the limits of visibility through the theodolite sight, the instrument man fixes the eyepiece cross-hairs on the pole (preferably at the base) held at

the apex of angle No. 1. After this, poles are located at every 100-150 m, proceeding from the apex toward the instrument. If the route location follows a line which has been drawn on a map, the

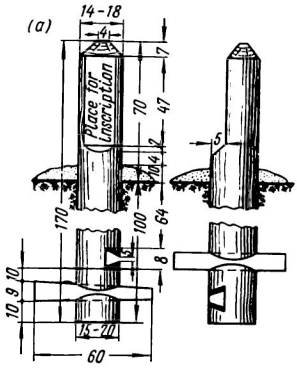


Fig. 155. Permanent survey post:
a—schematic view; *b*—general view

compass bearing of the first line (allowing for the magnetic deviation) is established, and the theodolite sight is aligned along this bearing.

As the instrument man—hereafter called the surveyor—progresses and the theodolite is advanced to the angle apices marked by the party head, the pole is removed from the previous apex and in its place is driven a wooden *marker stake* 25-35 cm long with a diameter of 3-4 cm. The theodolite is set up at the angle apex, levelled, and, by means of a plummet, accurately centered over the marker stake.

The angle of curvature is measured twice, with the circle in its "right-hand" and "left-hand" positions. The results of this measurement are entered in the instrument log book. To ensure the correctness of a measured angle it is checked against the compass. The divergence must not be more than 15 minutes, provided there is no magnetic anomaly.

A turning angle diagram is drawn in the log-book where the reasons for it are indicated. The angle of a turn and its direction are determined according to the magnitude of the measured angle read clockwise.

Guided by the instructions of the party head, the technical assignment and the local conditions, the surveyor selects the radius and determines all the elements of a curve. In difficult places—urban localities, when bypassing swamps, lakes and ravines, in proximity to road structures, overpasses and crossings, or if there are reverse curves—it is essential to check on the spot whether the curve radius has been correctly established. To do this the bisector and tangent positions are determined on the site, and the possibility of the road centre line passing through these points is ascertained. Using a theodolite, the route internal angle is divided in two and, having measured the bisector, the middle of the curve is found, at which point a stake is driven in, bearing the necessary inscription. On long curves the stations are marked out along the curve.

The apex of the angle is fixed by means of two posts set along each side of the measured angle. The theodolite sight is trained on the poles, then it is transitted, and at a distance of 10-15 m along this line a stake is driven in. In exactly the same way a stake is set along the second side of the angle. Appropriate inscriptions are made on each of these stakes. When the route is finally fixed these stakes are replaced with posts. The number and the magnitude of the angle, and also the elements of the curve, are communicated by the surveyor to the stationing group.

When the route joins an existing road or intersects another road, it is essential to measure precisely the angles between the two roads.

85. Marking Out the Stations

The stationing group comprises a technician and 3-4 labourers. The task of this group includes measuring the route and setting out station stakes (chainage); surveying the features of the landscape adjacent to the route and drawing them in the chainage log; measuring cross-sections of watercourses crossed by the route; surveying in detail railway level crossings, road intersections and marking out cross-sections.

Usually the route is measured once, using a steel tape 20 metres long. Measuring is carried out by two workers who align the tape between the fixed poles. The correctness of this measuring is checked with a range finder so as to avoid serious miscalculations (tape slip). Measuring accuracy must be at least 1/1,000 in flat country and 1/500 in mountainous country. At crossings of ravines, rivers and impassable swamps, distances are measured by triangulation. The

measuring of distances in inaccessible places is carried out jointly by the surveyor's and the stationing groups.

When laying out the route and marking out the stations in populated localities or in mountainous country there are cases when it is difficult to lay out the line and measure the route directly along the selected alignment. In such instances the survey line is diverted to a side so as to bypass the obstacles (Fig. 156).

Fig. 156. Setting out stakes when obstacles are encountered

To determine the horizontal projection of a route on land having a gradient of over 5°, it is necessary to introduce a correction, which is usually determined by means of special tables, or according to the formula

$$l = L \cos \alpha$$

where l = horizontal projection of the route
L = length of the route measured on the site
α = angle of slope of the land.

The angle of slope is determined with a theodolite or a clinometer. Where the angles of slope are comparatively small the horizontal route projection can be measured directly on the ground. For this purpose the person holding the end of the tape on the lower part of the ground raises it above the ground so as to make it approximately horizontal and then marks the projection of the tape on the ground. On steep hillsides, measurement in this way is carried out along separate sections of 5-10 m each, while on very steep hillsides it is necessary to use a straight edge with a spirit level attachment and a plumb.

At junctions of the measured alignment with another route or with its alternative it is permissible to use nonstandard stations (standard stations are 100 metres apart) of from 50 to 150 m. At each station a *marker stake* or *marker* is driven in 3-5 cm thick and 30-50 cm long with the station number marked on it. The stations are numbered in sequence, beginning with 0; the station number is written in ordinary pencil on the side of the marker facing the

beginning of the chainage. Just in front of the marker along the route line a small stake 3-4 cm wide and 15-20 cm long is driven in to ground level or a little above it. This stake, called a *point*, is sunk so that the surveyor's staff can be placed on it during levelling, and for this reason the top of the point must be horizontal and the point itself firmly driven into the ground.

At all characteristic breaks of ground surface along the route alignment, intermediate *plus points* are established, at which a point and a marker are driven in. An inscription is made on the marker showing at what distance in round metres from the nearest station the plus point has been installed.

The location of the plus points which define the position of the apex of the angle of curvature, of structures, level crossings, intersections, etc., is determined to an accuracy of 0.01 metre, and of those which define the position of the edges of river banks and of swamp boundaries—to an accuracy of 0.5 metre.

When marking out an alternative alignment, the number of the station (as a numerator) and the number of the alternative (as a denominator) are recorded in the form of a fraction both on the markers and in the log-book, for example, 137/2. The last stations before the junction curves are taken as the beginning and the end of an alternative, and not the apices of the junction angles. The station and plus point of the junction angle apex are determined exactly both for the alternative and for the main alignment, and are entered in the chainage log. After calculation of the curve elements, the latter and the chainage of the beginning and end of the curve are registered in the chainage log.

In flat and undulating country curves are not marked out since there is only a small difference between the elevation of points situated on the tangents and on the curves. The route is measured along the tangents taking into account the offsets, the chainage of the angle apices is determined precisely, after which the tape is located along the new direction and the end is moved a distance equal to the offset.

Using the surveyor's data concerning the magnitude of the angle of curvature, the selected radius and the curve elements, the head of the stationing group checks the accuracy of their calculation and determines the tangent points to the curve. At the points corresponding to the commencement and the termination of the curve, plus point markers are driven in.

If the road centre line at the curve differs in elevation from the line along the tangents, the stations, characteristic points and bisectors are set out along the curve using special tables for marking out curves. In broken and mountainous country circular curves are marked out in full, and also the transition curves, when neces-

sary. The number of points to be marked out is determined with a view to the curve length and the land topography.

Simultaneously with route measurement and setting out the stations, the land adjoining the route is mapped. The total accepted width of the strip surveyed is usually 100 m. On each side of the centre line a section 25 m wide is accurately surveyed with the aid of a tape line, while on the remaining part of the strip the width is estimated visually along the perpendiculars to the route. At the angles of curvature designed for bypassing obstacles, the survey strip is extended sufficiently to show on the plan the position of the obstacle which made the bend necessary. Important local features—urban localities, industrial enterprises, railways and motor highways, rivers, swamps, etc., which are beyond the limits of the hundred metre strip—are also entered into the chainage log with an indication of the visually determined distances.

All data concerning route measurement and mapping are entered in pencil in a *chainage log* with graph paper. In flat and broken areas the route layout is shown as a straight line, while in mountainous country a contour is traced. The adopted scale in the logbook is usually 20 m to 1 cm, but this may be increased for difficult areas (in populated areas, near railway crossings, etc.).

The position of the stations, plus points, arable land boundaries, angles of curvature, angle magnitude, radii and all curve elements are recorded in detail. When the road passes through urban localities, the house and fence outlines, street width, well positions, etc., are entered in the log together with notes on the total number of houses and the population of the community. When cutting across or running parallel to a telegraph, telephone or power line, the designation of the line, the number of wires, poles, masts, etc., are noted. Points where motor roads and railways intersect are usually shown to an enlarged scale.

The chainage logs also give the exact positions of bench marks and their descriptions.

At points where watercourses are intersected the cross-section is marked out precisely for levelling. If a route crosses a watercourse otherwise than at right angles, an additional cross-section normal to the watercourse is traced. When passing over an existing structure a detailed sketch is made on which the material of the structure, its basic dimensions and condition are given. In addition, the behaviour of the structure in relation to water discharge, the type of bed and abutment fortification are described. The setting out of cross-sections for subsequent levelling is also one of the duties of the stationing group.

Cross-sections are surveyed at characteristic points where a route passes over a hillside having a gradient of more than 20%, over

existing embankments and cuttings having side ditches or borrow pits; along broken land with uneven topography; along streets of urban localities; and also when it is necessary to provide drainage.

The width of cross-sections taken must be sufficient to check the possibility of using road-building machinery in construction of the road. In populated areas the whole width of a street up to the face of the buildings is surveyed.

Cross-sections are traced by means of an optical square or goniometer aligned at right angles to the centre line of the route. At the places where cross-sections are to be marked out a marker is installed on the route centre line indicating the station, plus point and the number of points which have been marked out to the right and left along the cross-section. At characteristic points on the cross-section markers are installed on which the distance from the route centre line is inscribed.

The chainage log serves as the basis for plotting the plan and profile of a route, compiling the list of straights and curves, and other important documents; therefore, all data should be entered in this log with the maximum of accuracy and clarity.

86. Route Levelling

Levelling along a marked-out route is carried out by a surveyors' group comprising two levellers and six labourers. This group carries out levelling of the route for compiling the profile, surveys the land cross-sections, surveys watercourses, thalwegs and their cross-sections, establishes new bench marks and lashes the route to existing bench marks.

Levelling of the route is carried out by means of a special instrument, a level, which is set up at mid-distance between successive staff positions in order to exclude the influence of earth curvature and refraction. The level must have a telescope with a minimum magnification of $\times 25$ and spirit level graduation corresponding to an arc difference of not more than 25″.

The distance from the level to the staff is established depending on staff visibility. On the average this is about 100 m. As a rule, the route is levelled in two stages, two levellers carrying out this work independently of each other at different times and using different staffs. Usually the first leveller surveys all stations and plus points, while the second one levels the reference points, surveys the cross-sections, watercourse sections and thalwegs. The distribution of the work between the surveyors is influenced to a great extent by the topography of the area. When levelling in broken or mountainous country on steep slopes it often becomes necessary to

set up arbitrary intermediate points called x-points, as otherwise, due to insufficient staff height, levelling would be impossible. For this purpose the staff bearers must have special supports—shoes for supporting the levelling staffs.

When crossing narrow ravines with steep slopes, the levelling of which may require many changes of instrument station, the elevation must first be transferred to the opposite side of the ravine, in order to correlate the elevations of the ravine cross-section in a closed traverse. Very steep ravine or hillside slopes are surveyed with an optical clinometer.

When crossing large watercourses (wider than 350 m) the elevations are transferred from one bank to the other by means of special marked (striped) staffs which are easily visible through the level telescope. The reading equivalent to the horizontal position of the sighting line is determined by calculation.

When crossing ponds, lakes or rivers which have a quiet current, a simplified method is employed: stakes are simultaneously driven in up to water level depth on both banks, and it is considered that the top points of these stakes have the same elevation. Having determined the level of the water on one bank, it is assumed to be identical for the stake on the opposite bank.

Sometimes, when the party has only one instrument, its height is changed at each station, and the variation in the calculated elevation differences should not exceed the double accuracy of the instrument. Instead of changing the height of the instrument double sided staffs can be used, but both of these methods are less accurate than separate levelling. A higher accuracy can be achieved when only one instrument is available by levelling first in one direction, and then in the opposite one. Usually the length of a section is chosen so that it may be levelled completely in both directions in one day. This enables any mistake to be immediately traced.

Whatever levelling method is used, it is advisable to complete a day's work at a bench mark or permanent point.

During the levelling work the leveller enters the staff reading in a special *levelling log*. The height of the instrument and the elevation differences are calculated in the field at each level station before going on to the next one. The records in the log are processed according to the rules laid down in geodesy handbooks.

All entries in this log must be made accurately. Incorrect entries and calculations must not be erased, but cancelled in such a way that it will be possible to read them.

Errors in the elevation differences for both instruments should not exceed more than 2 cm per 100 m, while the total permissible closing error over a section is $\pm 10\sqrt{L}$ cm, where L is the length of the section in kilometres.

Lashing of the route bench marks to the national standard bench mark network is done only where it is essential for design work. When lashing to absolute elevations (above sea level), double levelling is made with a total permissible error of $\pm 30 \sqrt{L}$ mm, where L is the section length in kilometres.

The magnitude of error with single or double levelling is determined for each kilometre of a route, for the route section covered in one day, and between separate bench marks.

After levelling of the entire route is completed, the magnitude of the total closing error is determined. Where actual closing errors in one of the cases given above exceed the admissible ones, the levelling is repeated.

For purposes of control in levelling, the route is lashed to all bench marks which were previously provided in the region through which the route passes. Before going into the field it is essential to find out the positions of such bench marks in the area of the route and obtain from the appropriate organizations their description and absolute elevations. In addition, the survey party sets out along the route a series of temporary and permanent bench marks. The temporary bench marks are usually wooden posts driven into the ground with a cross-piece at the lower end, stumps of felled trees, plinth walls of buildings, rocky projections in mountains, stone abutments of bridges, etc. These must be established at suitable locations so that the distance between them averages 3 km in flat country, 2 km in broken country and 1 km in mountains.

When crossing large watercourses, 1 or 2 additional bench marks are affixed on each bank. Bench mark elevations are established according to the 1st leveller's records. All bench marks are entered in a special *bench mark record.*

Temporary and permanent bench marks are positioned at points outside the boundaries of the anticipated construction sites.

When levelling cross-sections, the markers set out by the stationing group are taken as a guide, but in certain instances the leveller may himself establish an additional cross-section. For this purpose he must carry a tape or a measuring reel.

When surveying watercourse cross-sections or levelling thalwegs the leveller enters the levelling details into a separate log together with the other cross-sections.

87. Collection of Data for Structure and Drainage Design

The data necessary for the hydraulic calculation of structures and the design of drainage are usually collected by a special survey group concerned with structures.

The tasks of the group include:

(1) Determination of the boundaries, areas and lengths of the catchment areas.

(2) Determination of the average thalweg gradient, side slope gradient and the gradient in the vicinity of the structure.

(3) Determination of water flow conditions from the catchment area, particularly, the intensity of water absorption by the catchment area soil mantle, the existence of vegetation, agricultural lands, inhabited localities, swamps, lakes and dams which influence the water flow conditions.

(4) Survey of the cross-section normal to the flow of the river if the route does not cross the watercourse at right angles.

(5) Survey of cross-sections and the drawing of sketches of existing structures; determination of water discharge through these structures, and their condition.

(6) Collection of data for planning special drainage structures, drain and intercepting ditches, evaporation basins, velocity-breaking steps, flumes, chutes, etc.

(7) Performance of simple hydrometric jobs.

This group begins work in the field after the stations have been staked out, bench marks fixed and levelling finished.

Firstly the catchment area dimensions for the chosen route are determined. This is done either on a map or by measuring on the site. Catchment areas from 0.5 to 3 km^2 are calculated according to maps at a minimum scale of 1:50,000, and those of an area from 3 to 20 km^2 at a minimum scale of 1:100,000. Areas of larger catchment basins are determined according to maps of an even smaller scale, provided the catchment area boundaries can be determined on them. Areas under 0.5 km^2 are determined by field survey.

Catchment areas are determined on a map by first establishing their boundaries, which will normally coincide with the watershed contour lines. On a contour map it is easy to establish these boundaries. If a contour map is not available, the catchment area boundaries are marked approximately, at the middle between the sources of streams.

For compiling a catchment area plan different methods may be used (Fig. 157). For instance:

(1) Small catchment areas in open country are sketched in the field and the catchment area boundary direction is established by means of a compass. The distance is measured with a pedometer or simply by pacing (catchment area No. 1).

(2) Large catchment areas in open country are surveyed by means of a theodolite. One proceeds along the fixed poles and catchment area boundaries with a theodolite, measuring the distances with a range finder or, less frequently, with a tape.

(3) To establish the catchment area boundaries in an enclosed area, it is necessary to execute a theodolitic traverse over the main and lateral stream beds to the watershed; the distance between the set points is measured by means of a tape, by pacing, etc., or by graphical analysis of the plotted traverses (catchment area No. 2).

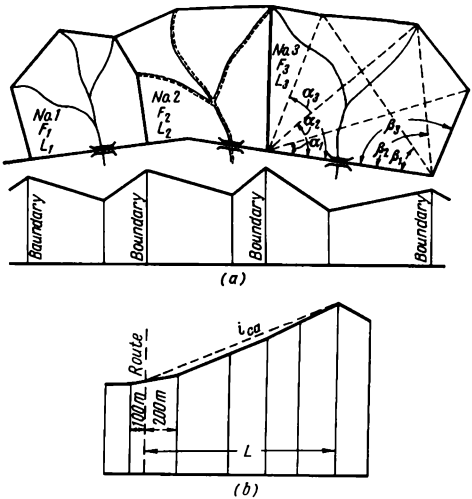


Fig. 157. Catchment area survey:
a—catchment area plan; *b*—determination of longitudinal gradient

(4) Small catchment areas, whose boundaries are visible from the route, can be surveyed directly by simple triangulation using one or two theodolites. On the basis of site inspection, poles are set out at characteristic points of the catchment area boundary. The theodolite is placed at the watershed point of the route and sighted on the poles successively, and the angles α_1, α_2, α_3 are measured. Then the theodolite is moved to the next watershed point of the route and the angles β_1, β_2 and β_3 are measured (catchment area No. 3).

When drawing the catchment area plan, the areas of lakes, ponds and swamps situated within the boundaries of the catchment area should be determined in order to confirm the coefficients of the storm

water discharge formula. At the same time the possibility of an increase of the rated discharge owing to water runoff along the drain ditches must also be taken into account. The catchment area plan is used to find the distance from the structure to the catchment area centroid.

The length of the catchment area is measured along the main river bed from the point where it is intersected by the route to the catchment area boundary. For this end a map or survey plan of the catchment area is used. The average catchment area gradient is determined by levelling along the main stream bed from the route crossing to the watershed by means of a contour map or a tacheometric traverse. The gradient adjacent to the structure is determined between points sited in the stream bed, one of which is placed 200 m upstream of the structure and the other 100 m downstream. To determine the soil category in respect to permeability shallow pits are dug to a depth of 0.5 m. For this end the upper soil layers occurring at a depth of 20-30 cm are the most important. In preliminary surveys 1 or 2 pits are made in each catchment area, while during detailed surveys one pit is made for each square kilometre, so that their total number will be minimum 2 and maximum 10.

If there exists a road or railway structure over the river near the planned crossing, detailed data characterizing its state and conditions of service must be collected.

If there are bridges and culverts along the route, it is necessary to investigate them, draw a longitudinal and cross-sections, determine the condition of the structure and vehicle clearances, and decide whether it is possible to use them. For structures situated away from the route a sketch drawing may be made indicating the dimensions of the opening, the vertical clearance over the river bed, a description of the opening behaviour and the type of protection of bank and bed.

When surveys are made in areas for which there is no exact data on the amount of runoff, information may be acquired from local meteorological stations on rainfall rates and runoff. In mountainous country where mud flows of alluvial deposits may occur the crossed streams are inspected so as to establish the most favourable locations where structures can be designed to discharge such mud flows.

In broken and mountainous country, depending on the natural drainage, it will be necessary to provide such facilities as intercepting and discharge ditches, also velocity-breaking steps, checks, chutes and flumes. For this purpose the centre lines of the proposed ditches are levelled, cross-sections are surveyed, and at points where structures are to be sited surveys of the surrounding terrain are made.

For the design of evaporation basins in flat country, convenient depressions in which they can be arranged should be sought. The openings of drainage installations and special structures are computed during the field work in order that the design solutions reached be compared, checked and adjusted on the site.

88. Setting Out the Route

Between the execution of the engineering surveys and the commencement of construction work much time may pass, during the course of which turning angle and station marks are lost. This causes difficulty in establishing the route to be followed in construction of the road. Therefore during the survey process it is essential

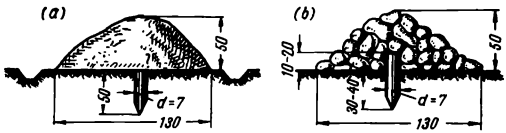


Fig. 158. Keeping marked-out points intact:
a—earth mound; b—cairns

to set out the route on the site permanently by using special signs which are installed at the beginning, at turning angles along the route, and on long straight stretches. The capacity of these signs is filled by sunken stakes, special wooden posts, cairns or various permanent objects. Sunken stakes 6-7 cm thick and 40-50 cm long are driven in flush with ground level and over them earth mounds 50 cm high are erected (Fig. 158*a*). Around the mounds small ditches are dug to a depth of 10-15 cm. If stones are available, they are piled up over the sunken stake (Fig. 158*b*). Where a route passes over an existing paving a stake is driven below the paving level, and on roads with metalled surfacing small holes measuring 5-10 cm are made at set points, which are filled with cement.

To preserve the metalled road surfacings it is preferable to place the route markers on the shoulders.

Sunken stakes must be lashed to at least two permanent local objects (the corner of a building, telegraph or kilometre posts, etc.). The location details of the sunken stakes are entered into a special log or *route setting-out record.*

For setting out the route wooden posts having a diameter of 15-20 cm and a length of 1.7 m are frequently used. These are driven into the ground to a depth of 1 m. At the lower half of the post

a cross-piece made of boards is fixed. Around the post at a radius of 1 m a ditch is dug. These posts are also lashed to local permanent objects. The scheme of coordination is entered in the setting-out record.

Inscriptions are made on the marker signs with oil paint. These indicate the body carrying out the survey, date, sign number and what is located, viz., angle of curvature, commencement of the route, etc. On long straights the signs are installed at intervals of 1 to 3 km so that from each of them the two neighbouring ones are visible.

The signs are placed exactly on the route centre line, for which purpose the direction is set out with the aid of a theodolite. The route centre line is marked on the post with paint, or a nail is driven into the top of the post. It is essential that the terminals of the route alternative be reliably marked in difficult relief conditions. Where a route passes through populated localities, in addition to the usual marking of the route and angles of curvature, etc., it is necessary to mark the location of stations on fences and building foundations, indicating the distance from the route centre line.

Where large watercourses are crossed three signs are installed on each bank on each marked-out cross-section. Of these, two signs should be within the limits of the area flooded by high water and one sign at high water level. Crossings over small watercourses are set out by installing one or two signs on each bank.

89. Mapping Complicated Sites

To select the route direction and draw up the design on exceptionally difficult sites, contour plans are made. A plan is usually drawn using a theodolite-tacheometer or plane-table. The plan is drawn to a scale of 1:500-1:2,000 with contour line intervals of 0.5-2.0 m, depending on the purpose of the survey, the dimensions of the plan, nature of land topography and situation.

A contour plan is usually made in those instances when the route sections pass through broken country with large longitudinal and transverse gradients or in unfavourable hydrogeological conditions (talus, landslides, sink holes, frost heaves, etc.); also when the route crosses a railway line, motor road, ravine or alluvial wash-out, where demolitions may be necessary, and, in particular, where the route crosses large and medium-sized watercourses.

When surveying, the contemplated route is taken as the basis, serving as the basic network for the tacheometric survey. If the route has not been located, the most convenient line set closest to its probable position may be selected for this purpose. If the width of the survey zone is less than 300 m, the survey may be done directly

from the route or the chosen line. First the angles along the route must be measured, the stations set out and levelling performed, including that of turning angle apices. The zone is surveyed using what is called the *polar method*, when from each station sightings are taken by means of a tacheometer on both sides of the route. Two technicians and a team of 3 or 4 labourers are employed on this work. One technician makes out a rough sketch, i.e., sketches in the log the situation and relief and selects the point for setting up the staff, while the other works at the instrument. Where essential, from the selected base line an additional run may be extended for elucidating details situated away at a distance of up to 0.5 km. The lengths

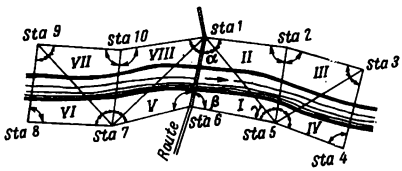


Fig. 159. Reference grid for river survey

of such runs and the elevations of the additional stations may be determined with a range finder and a vertical circle used for measuring slope angles.

For the survey of an area a reference grid is first set out either as a closed traverse or a chain of triangles. A closed traverse is usually employed in open flat country or when the most important details are situated mainly on the periphery of the area. The length of the sides of the traverse is usually within 150 to 1,500 metres. Some of the traverse angles may coincide with the route turning points.

The total error in the length of the closed traverse must not exceed 1/1,000 of the perimeter and it is then distributed in proportion to the length of the sides. A reference grid in the form of triangles (Fig. 159) serves as a more accurate basis for surveying.

The length of the traverse sides is measured twice with a steel tape, while all the internal (on the right-hand side) angles α, β and γ are measured using a theodolite. Errors in the measurement of the angles must not exceed a magnitude of $\Delta = 1.5\sqrt{n}$ minutes, where n is the number of angles measured.

The grid errors are distributed consecutively for every triangle and therefore they do not accumulate. Usually a section of the route is included in the reference grid as the base of one of the triangles. Sometimes a side of a triangle is taken as the base provided that on

setting it out on site it can be accurately measured. The base length is measured twice with a tape to an accuracy of 1/2,000.

The elevation of the reference grid points is determined by levelling, and the error is distributed as in a closed traverse.

If a trigonometric point is situated close by, the reference grid is first lashed to it and then the coordinates of all the grid apices are calculated. If there is no trigonometric point, any permanent point in the locality is adopted as the origin of coordinates, and this is then lashed to the reference grid and shown on the plan.

The plan and the topography are plotted from the reference grid stations by means of a tacheometric survey and by individual measurements using either a tape or a measuring reel.

The contour map is drawn on the basis of the data of the tacheometric survey logs.

90. Soil Investigations

The route direction, the design of the roadbed, the carriageway structure, the type of reinforcement and the drainage are selected with a view to local soil conditions.

The survey group usually includes a geological engineer or a soil specialist, a geologist and a laboratory assistant to carry out the soil investigation work.

The geological party helps the party head to assess the soil conditions when locating the route, they dig trial holes and pits, bore holes for determining the soil and hydrogeological conditions, carry out laboratory investigations of soil samples and compile detailed specifications giving the description of the soil, geological and hydrogeological conditions of the line of route, as well as corresponding design recommendations.

The soil conditions along the route are investigated in the same general manner as during the reconnaissance survey, but the investigations should be considerably extended for the detailed survey.

The depth of soil investigated should be sufficient for obtaining a clear notion of the main soils to be used for constructing the roadbed. In normal conditions it is necessary to examine the soil to a minimum depth of 2 m, and, in the places where the design requires the excavation of cuttings, exploration should continue to 1.5-2.0 m below the future base of the cutting. For embankment construction in flood areas, where consolidation of the natural ground under the imposed weight of the embankment is anticipated, it is necessary to investigate the soil to the depth of possible consolidation.

Bore holes are drilled at all characteristic points of the relief: at watersheds, hillsides, depressions, thalwegs and ravines (Fig. 160). When determining the location of bore holes account is taken of

variations of ecology, which usually characterizes a variation in soil conditions. For each kilometre of route length not less than 2 bore holes are drilled. For the investigation of borings a special log is kept where all the data concerning the structure of the soil section, the texture, composition, density, porosity and the colour of individual soil layers are entered. Records should also be made of the natural pore-water content in individual layers, the depth of the water table and the rate of water inflow into the bore.

A drawing of the soil section is made in the log showing the thicknesses of individual soil strata. The more characteristic sections are photographed. Samples of soil having dimensions 0.2 × 0.2 × 0.2 m are taken from separate strata for laboratory investigations. The

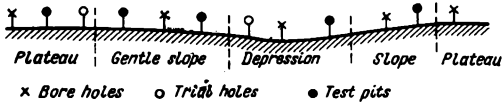


Fig. 160. Location of bore holes and test pits in relation to land topography

place where every sample was taken is recorded, and the sample is labelled showing the depth at which it was extracted and the number of the bore hole. Sometimes undisturbed soil samples 1.0-1.2 m long and 0.2 × 0.1 m in cross-section are cut from the side of the bore hole and are then placed in special cases.

Test pits and trial holes are dug between the bore holes in order to glean additional information concerning the variations of soil conditions. Test pits are of 0.7 × 1.3 m in plan and are dug to a depth of up to 1 m; trial pits are respectively 0.25 × 0.75 m in plan and 0.5 m deep. If a trial hole reveals a substantial departure of the soil section from that revealed in an adjacent bore hole, then it is widened and deepened to bore-hole size. Trial holes are dug at intervals of 250-300 metres.

All bore holes, test pits, trial holes and auger borings are lashed to the chainage of the route and are registered in the chainage log. In deep cuttings (up to 10 m) where there is no ground water, bore holes are drilled every 100-150 m, with not less than 2 bores per cutting. For cuttings deeper than 10 metres, and also for all cuttings having unfavourable hydrogeological conditions, holes and auger borings are staggered on both sides of the route, their number being sufficient to assess fully the character of the hydrogeological conditions, but at least 3.

On sections where high embankments (over 10 m) are to be constructed the bore holes are drilled at intervals of 50-100 m, and the

investigation is continued to an average depth of 3-4 m below the surface. In places where minor structures are to be sited small-diameter drills are used for boring, having diameters of 50, 60, 78 and 89 mm. For bridges having a minimum span of 10 m and for short culverts under low embankments one boring is made to a depth of 5-8 m, while for culverts 25-40 m long 2 or 3 borings are drilled. The depth of borings at places where culverts are to be installed depends partly on the height of the future embankment: for an embankment height of up to 12 m the bore hole depth is 8-10 m, and for higher embankments the depth of the bore hole is made approximately equal to the intended height of the embankment.

The locations of future borrow pits are also surveyed. The number and the situation of the borings are determined according to the structure of the roadbed and the depth of the borrow pits. At the same time the dimensions of the pits, data relating to the soil, the depth of individual layers, the soil structural category and the conditions of excavation are established.

In mountainous regions geological surveys are of the greatest importance. In this case all natural exposures are investigated, and not less than 3 bores are drilled for obtaining geological sections. Two or three such sections are procured for parts of the region that are characteristic from a geological viewpoint. Route sections along which landslides, slips and strata crumpling glissades are possible are investigated in great detail. When surveying an area subject to landslides the areas of the slides are determined, the structure of the landslide strata is investigated, as well as the causes which have led to the development of the landslides, and, finally, the possible means of prevention are formulated.

The plan of the landslide is determined by means of a plane table or a tacheometer to a scale of 1:1,000-1:2,000 with contour lines every 0.5-1.0 m. On the plan shown in Fig. 161 all the characteristic elements of a landslide relief are indicated: strippings, fissures, upthrusts, points of ground water emergence, and places of geological prospecting (dug holes, cross-sections, lines of geological sections and landslide bench marks). A grid of sections is traced over the landslide areas at their characteristic points, which are lashed to the route.

The depth of the dug holes and auger borings depends on the depth of the slip surface, extending 1-2 m below it. Since water can accumulate in dug holes which can later percolate into the body of the landslide, boring is usually preferred. If, however, the holes are dug, they are propped, and after work they are immediately backfilled and thoroughly compacted. The soil is extracted from the dug holes in the form of large undisturbed samples.

A log is kept of the dug holes or borings, showing the thickness of the rock layers through which the holes were sunk and the levels at which water appeared. For laboratory investigations samples are taken from each layer. On the basis of this work geological sections are drawn on which all the soil layers, ground-water levels and slip surfaces are indicated.

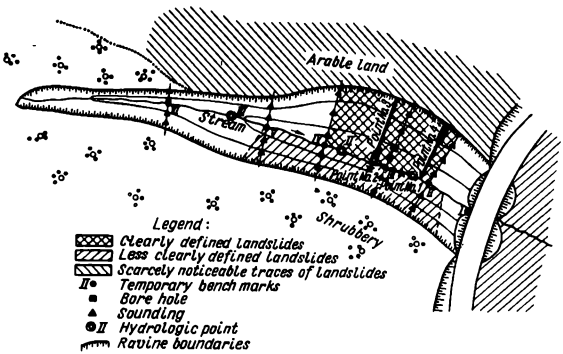


Fig. 161. Plan of a landslide area

During the field work the geologist should inspect the bore holes and the samples of soil, establish the specific depth of soil layers and, if necessary, alter the disposition and depth of bore holes, and arrange for additional holes.

When boring it is important to establish the depth of the water table and determine the rate of flow. The latter is done by means of trial pumping. The direction and rate of ground water flow between dug or bore holes can be determined with the aid of indicators soluble in water, which are introduced in the bore holes at the highest elevation and detected in the lower ones. A solution of sodium chloride or lithium chloride is used as an indicator, which is detected in the water by chemical analysis. A simpler and more reliable method is to use dyes, e.g., fluorescein (aniline dye), which can be detected in water even in very small concentrations.

A more accurate method of determining the rate of flow involves the use of an electrolyte indicator, for example, ammonium chloride. An anode is immersed into one of the bore holes and a cathode into another one. An ammeter or galvanometer is included in the circuit, and the deviation of the instrument pointer when the circuit

is closed by the electrolyte indicates the flow of the ground water. The instrument can be provided with a self-recording pen that registers the time.

Recently use has been made of radioactive indicators which are introduced into one of the bore holes and detected in the others by means of a Geiger counter.

Simultaneously with the survey of a landslide the conditions of surface water runoff and the possibility of its percolation into the body of the landslide are investigated. The catchment area from which the water runs off towards the landslide is examined, and the slope, conditions of flow, permeability of the soil, places of water accumulation and fissures are also determined.

In a special explanatory note the landslide area is described, as well as its geological structure, hydrogeological conditions and the mode of development of the slip. The possible ways of countering the landslide are also described in the note.

A diagram of any substantial landslide is drawn to a scale of 1:1,000 to 1:2,000 with contour lines every 0.5-1.0 m. Based on the land survey, the nature of the movement and the area of accumulation and shifting of demolished rock are established. According to bore hole data and the exposures the nature of the soil area causing the fault, the depth of the active layer (hanging wall) and the type of underlying strata (foot wall) are determined.

By means of special bench marks the rate of slip and the intensity of material accumulation are determined. In addition, the conditions of water inflow and filtration through the body of the talus are elucidated.

If the road is in service the road officials collect information on the development of sliding, and on material accumulation in the course of a year and during separate periods. On the basis of this information the road officials plan measures for fortifying the talus or for erecting barrage and diverting structures.

Upon carrying out surveys in karst (sink hole) country the degree of development of the karst processes is established. Careful visual inspection of the land will reveal karst holes, faults, folds and fissures, as well as likely periods of their formation and development. The conditions relevant to surface water runoff are also investigated (the amount of water accumulation over the catchment area, jointing of the surface, the effluence of ground water). In the neighbourhood of the route dug and bore holes are driven. Geological sections are used to determine the nature of the soil strata and the degree of jointing of individual layers, as well as their solubility in water. When boring the holes the ground-water table, hydrologic conditions and the sources of inflow are determined. The material obtained as a result of the geological survey should be sufficiently

detailed to make clear the possibility or impossibility of building the road in the given region, and to substantiate the design solutions for eliminating or reducing the karst processes.

Ever wider use is being made in road surveys of geophysical methods of soil investigation, namely, resistivity and seismic methods. The electric resistivity method is based on the difference of resistance of various soils to the flow of electric current. An electric potential is applied between electrodes immersed in the soil, and intermediary electrodes measure the average resistance of the soil between them. The specific resistance of soil is less than that of dense rock: thus, if for clay the resistance is 3 to 50 ohm/m, then for limestone it is 600 to 500,000 ohm/m, and for granite still higher—8,000 to 6,000,000 ohm/m.

The degree of saturation of the material greatly influences the results obtained from these measurements.

The seismic method of soil investigation is based on the difference in the propagation velocity of elastic waves in soils having various density. Thus, for soils these velocities are in the range of 500 to 1,500 m/sec, and for rock they increase sometimes to several kilometres per second.

The propagation of waves created by a test explosion is measured by a seismograph located at some distance from the point of explosion. The seismograph receives the waves propagated in the soil top layers, and later those reflected by rock occurring at a certain depth. It is now possible to calculate the depth of rock occurrence.

Upon surveying in saline regions the number of bore holes and pits is increased because the nature of soil salinity is very sensitive to changes in microrelief. In depressions the degree of salinity is usually greater than on ridges and slopes. Samples taken from bore holes at all the separate characteristic levels are selected for subsequent laboratory investigations.

Bore holes and trial pits should also be excavated in places of prospective borrow pits and drain ditches in order to determine the possibility of using the soil for filling in the embankments. When laying a route through saline soil with a high degree of salinity the geologists have to locate suitable sources of salt-free material for use in the embankments.

According to the bore hole data a soil profile is compiled, in which the traced ground-water levels are indicated.

91. Basic Safety Rules for Highway Surveys

When carrying out highway surveys it is necessary to observe safety and sanitary regulations and to provide the survey personnel with the required clothing, shoes, individual protection and first aid kits.

During work the heads of the survey parties should systematically train their workers and engineering personnel in safety methods.

In the field it is necessary, first of all, to provide adequate sanitary conditions. In the camp a boiled-water tank, washing facilities and soap should be provided. During work in dense forest and in regions which are infested with mosquitoes and midges, it is necessary to provide every member with a mosquito net, insect repellents and medicaments for applying to exposed parts of the body. When working in regions where infection with malaria or encephalitis is possible the members should be inoculated and receive special medical instructions.

When surveying morasses and floating bogs precautions should be taken to prevent the men from becoming trapped. The men should keep close to each other in order to help their companions if necessary. Every member must be supplied with a ranging pole at least 2 metres long for checking the firmness of the surface crust.

When fording rivers it is necessary to investigate beforehand the river, its depth and the velocity of its current. It is also necessary to ascertain who of the members can swim. Initially the ford should be tested by a good swimmer. Fords can be crossed without protection only if their depth does not exceed 0.6 m with a maximum current velocity of 3 m/sec, and 0.4 m if the velocity is above 3 m/sec. Crossing on horseback is allowed when the depth of the ford is less than 0.8 m. With deep fords or with high current velocities the fording of rivers should be attempted only with the men tied to a rope which is securely anchored ashore.

When surveys are carried out in dry seasons in forests or in steppe country great care must be taken with fires. The fires should be built most carefully and extinguished when not needed. Combustible and inflammable materials should be stowed in closed containers at a minimum distance of 100 m from buildings.

When the party is moving through a wood or undergrowth precautions should be taken to prevent the men from injuring each other with the sharp ends of their poles, tripods, crow bars, hatchets, etc.; for this reason the men must follow each other at a minimum distance of 3 to 5 metres.

When cutting an opening in a forest the trees which have to be felled should be brought down between neighbouring trees in order not to obstruct transverse openings and roads. First, a tree should be cut from the side towards which it is to fall at a height of 1/2 to 2/3 of its trunk diameter, the tree being cut about one third through. Then from the opposite side, slightly above the first cut, the tree is sawn until it starts to incline. At the moment when the tree falls all the men should stand away from the stump at a distance

of at least 3 to 4 metres. Attention should also be paid to the correct cutting off of branches from the felled trees.

When surveying existing roads with a heavy traffic flow, traffic controllers armed with red and yellow flags should be posted at both sides of the party and at a distance of 50 to 100 metres therefrom. Survey instruments and appliances should not be left lying on the carriageway.

When ranging a line along the road centre line, the poles should be placed in special moveable supports. It is forbidden to place crowbars, metal tubes, etc., instead of poles, in order to avoid accidents.

Upon surveying railway crossings a man should be posted to watch train movements and give warning when a train approaches. When visibility is poor during bad weather work should be stopped.

When intersecting overhead power lines it is forbidden to measure the height of wire suspension by means of a tape line, pole, staff, etc. This height should be found indirectly by means of an angle gauge and a staff.

For carrying out hydrologic work men should be selected who are able to swim and row. When measuring depths or current velocities by means of flowmeters and doing other similar work, it is necessary to comply with special regulations. The hydrometric stations should be provided with life-saving equipment.

During engineering and geological surveys, when boring, driving tunnels and prospecting, the relevant safety regulations must be observed. In view of the importance of safety regulations for this work, every man must be properly instructed. All work should be carried out only in the presence of engineers and technicians. The safety regulations must be displayed in a prominent place on the boring rig.

The engineers and technicians should be familiar with safety regulations and see that the workers know and obey these rules, since the heads of survey parties and groups bear all responsibility for accidents.

92. Office Processing of Survey Materials

The traverse, chainage, levelling, tacheometric and other logs are processed every evening after field work. After processing and checking the data, the route plan and profile, cross-sections, plans of individual difficult places, diagrams of existing structures, etc., are drawn. According to the log data lists of straights, curves and turning angles, bench marks, existing structures, etc., are compiled.

When surveying in difficult conditions, special draughtsmen are included in the party for carrying out office processing.

If the survey party has a camera it is worth adding to the explanatory notes photographs of separate sections of the route, intersections of roads and rivers, existing structures, quarries, borrow pits, land relief, etc.

The party may leave the survey region only after completing all office processing and after being sure of the precision, correctness and adequate quantity of their collected data. All marks placed by the party in the field (bench marks, angle apex and route fixing posts) are entered into an inventory and handed over to the local authorities for safekeeping.

All the field materials of detailed surveys are submitted fully processed, and well and accurately drawn.

The results of the survey are represented in the form of the following materials:

(1) a topographic map with route alternatives and an indication of quarry locations;

(2) an explanatory note;

(3) field records;

(4) a plan of the route to a scale of 1:10,000 (for mountainous regions 1:5,000);

(5) plans of individual difficult sections to a scale of 1:5,000-1:500;

(6) a profile;

(7) cross-sections of the land at characteristic points;

(8) diagrams of existing structures;

(9) calculations of structure openings;

(10) data concerning the soil, engineering and geological investigations, and the results of deposit prospecting;

(11) documents on the approval of the chosen route alignment.

CHAPTER 15

THE HIGHWAY TECHNICAL PROJECT

93. Scope of Technical Project

The technical project, which is based on the approved project report and on the materials of the detailed engineering survey, should include a full description of the plan, profile and cross-sections of the route, the design of the roadbed and the carriageway in various geophysical conditions, the dimensions and design of structures and the precise quantities of work. The accepted design solutions are substantiated by engineering and economic calculations.

The requirements which the technical project has to meet depend on the class and the purpose of the contemplated road. The technical projects for roads of higher classes are very complicated and are carried out by a great number of engineers and technicians. Projects for roads of a lower class and of a reduced length are much simpler and smaller in volume.

In the course of compiling a technical project the maximum use is made of existing standard drawings and solutions, which facilitate and reduce the cost of designing.

Special attention should be given to economy, avoiding excesses in projects and estimates. For this purpose use should be made of progressive design solutions, and provisions made for the maximum use of local building materials, for complete mechanization and progressive methods of work.

94. Designing Road Plan, Profile and Cross-sections

The location of the road in plan is determined in the process of survey and when working out the project report. When compiling a technical project the route given in the project report is taken as the basis, and its location is finalized in the difficult places after detailed engineering investigations. Here several alternatives of the route may be investigated and compared, the one having the best engineering and economic indices being chosen. Such a comparison cannot always be accomplished under field conditions, since every alternative has to be designed precisely, including structures, and the quantities and cost of work determined. This is done when the technical project is being worked out in the office.

When designing a road in undulating or mountainous terrain, if a contour map is available, new alternatives may be worked out

which improve the original road location. Upon examination of the route plan the possibility of increasing curve radii is investigated, the situation of transition curves is checked and it is established on which curves a super-elevation should be inserted.

When designing a route in a built-up or forested area she required minimum sight distance should be provided, In a broken or mountainous area, when curves of small radii coincide with cuttings, the adequacy of the sight distances should also be checked.

To assess the quality of route location, engineering and economic factors are used to compare the route alternatives. A list of these factors is given in Sec. 104.

The profile is planned in accordance with the location of the line determined in the course of office processing of the data.

On the basis of the data relating to the class of the road, soil and climatic conditions, the depth of ground water, and the design of the carriageway, the recommended *elevation difference of the roadbed is established.*

In various natural conditions the route can be divided into individual homogeneous sections, and for each section the recommended difference between ground and grade elevations is established. The profile at structures such as bridges and culverts is designed after hydraulic calculations of the dimensions of the openings and the minimum height of the embankment.

When designing the profile on sections coinciding in plan with curves of small radius it is necessary to ease (decrease) the longitudinal gradient. Curves of small radius at the end of a downgrade are inconvenient for traffic, since vehicle speed has to be reduced along the whole downgrade.

In designing account should be taken of the possible simultaneous horizontal and vertical curvature of the route. If this is not done, then a change of gradient in a deep cutting may coincide with a horizontal curve of small radius, which would greatly reduce visibility on the road.

In broken land with small hills and depressions it is preferable to design the profile along an intersecting line with longitudinal hauling of the soil from cuts to fills.

In flat and gently undulating country the grade line is designed along an enveloping curve, exclusively on embankments. This does not mean that one has necessarily to follow strictly all the variations of relief, but efforts must be taken to make the grade line easy and smooth. The distances between breaks in the grade line should be so set as to permit location of the vertical curves. In very flat country, in order to provide for proper water drainage an undulating profile can be designed. However, such a profile is not convenient for traffic movement. The quality of the grade line is assessed by the quantity

of earthworks, the smoothness of the line and the proportion of the total route having maximum gradients.

The roadbed cross-sections for heights of embankments or depths of cuttings up to 12 m are planned according to the rules set forth in Chapter 7. The cross-sections are drawn to a scale of 1:100-1:200.

The carriageway cross-sections are worked out according to the design of the pavement with a view to local soil conditions and the building materials employed. The pavement design and the deformation modulus of the roadbed layer to be analyzed are shown on the profile drawing for characteristic sections.

Where high embankments and deep cuttings are planned, and on steep hillsides, the slope of the roadbed sides is set after checking their stability.

Where the hillsides are steeper than 1:10 the grade line is plotted on all the cross-sections surveyed during the field work and the design of the roadbed and the type of slope reinforcement to be used are determined. In this case the quantities of earthworks are calculated according to the cross-sections.

If the road passes through a swamp, the design of the cross-section is selected depending on the road class and the peat density.

95. Determination of Work Quantities

When drawing up a technical project the dimensions of all structures and the quantities of all kinds of work are determined, these being calculated in special lists and later combined in a summary list of *work quantities.*

The quantities are determined for all the principal parts of the technical project, namely, the preliminary work, construction of the roadbed, structures, carriageway, communications, civil structures and buildings, road signs and accessories, equipment of service buildings, and temporary structures.

The preliminary work includes the restoration of the route, the appropriation of the right-of-way, the demolition of buildings and payment of compensation, the felling of trees, uprooting of stumps, relocation of communication lines and tramway tracks, etc. The quantities of all this work are calculated in special lists.

The work relating to the construction of the roadbed includes earthworks and also fortification work, the arrangement of drainage including drains, chutes and velocity-breaking steps, the reinforcement of ravines and measures against frost heaves. The main earthworks are calculated in a list by stations, and then the quantities of earthworks per kilometre of the route are summarized, indicating the soil groups, haulage distance and methods of excavation. Additional earthworks, in case of flat or broken country, are assessed

as a percentage of the main earthworks. The additional earthworks comprise the arrangement of drain and hillside ditches, the filling of holes after uprooting of stumps, the straightening of river beds adjacent to minor structures, etc. To determine the percentage of additional works, data from similar completed projects can be used, or the quantities for several typical route sections are calculated and then extrapolated for the whole length of the route. The additional earthworks average 3 to 5% of the main ones. The fortification works consist in the stabilization of the roadbed slopes, river beds, etc., the erection of velocity-breaking steps, chutes and the stabilization of ravines. The type of stabilization is determined by the discharge and velocity of the water.

The embankment and cutting slopes in silty and fine sands are protected by continuous turfing with any elevation difference. With sandy loams and silty soils, when the difference between ground and grade elevations is less than 2 m, these slopes are stabilized by sowing grass; where the difference is from 2 to 8 m—by tessellated turfing and sowing grass within the spaces, and where it is over 8 m—by continuous turfing.

The slopes of embankments up to 8 m high in loamy and clayey soils, and of cuttings 2 to 8 m deep, are stabilized by sowing grass, and, when higher or deeper, by tessellated turfing and sowing grass.

The quantities of work for the provision of velocity-breaking steps, chutes, aprons and drainages are calculated according to the detailed construction drawings. All structures planned along the route are recorded in a summary list of structures.

Work quantities for the construction of carriageways are calculated in a carriageway list; additional work quantities relating to the construction of the carriageway on curves (super-elevations and widenings) are recorded in a special super-elevation list.

The quantities involved in the erection of buildings (road administration and division headquarters, repair shops, hotels, service stations, etc.) are determined according to the projected distribution of these buildings along the road, their number depending on the class of the road and the adopted methods of road maintenance.

The quantities of work for the erection of indication and warning signs, kilometre and station posts, safety fencings (safety posts, parapets), railway crossings and slip roads also include the planting of vegetation for aesthetic and snow-protection purposes. Data relating to all of this work are recorded in a list of road signs and guards, of road stretches subject to snow drifts, and of designed belt plantings.

The equipment of the buildings along the line and the number of temporary structures are determined in relation to the designation and length of the road, the duration of work and the disposition of inhabited localities.

96. Composition of the Technical Project

The technical project of a highway or a major structure, submitted for approval, consists of a series of documents and auxiliary materials which are kept in the archives of the project organization and part of which are later handed over to the contractor for compiling the working drawings. The auxiliary materials include the work quantities and technological, hydraulic and statical computations. The documents are divided into two groups so as not to overload the project with documents not required for its examination and approval.

The following is included in the technical project:

1. The importance of the road and a description of the chosen location, or of the site for a bridge crossing; a topographic map of the area, a plan of the route and also plans of difficult places.
2. The profile along the adopted route, the design of the roadbed and pavement, the projects of road intersections, the lists of main construction work quantities.
3. Plans of crossings on which the bridges, approaches and regulating structures are shown together with the calculation of bridge openings; detailed drawings of bridges and geological cross-sections.
4. Design drawings and statical calculations of complicated engineering structures—retaining walls, snow safety fences, etc. For minor structures the results of hydraulic calculations are submitted together with a summary list of structures.
5. Information detailing the project report in respect to the buildings for the operation service, materials and work organization.

The technical project includes all the main documents of the project report, detailed according to supplementary data, as well as additional materials. A detailed list of the documents included in the technical project and the rules for its presentation are given in the departmental instructions and recommendations. All the materials of the technical project are included in separate volumes, with 4 or 5 copies of each volume. The supplementary materials consist in the main of the original field data, lists of work quantities and auxiliary calculations. All the documents of the technical project must be signed by the head of the design office, the chief engineer of the project, and the engineers who compiled and checked the project.

The approved project report in two-stage or technical project in three-stage designing is used for compiling the working drawings.

The quality of a project depends in the main on the extent to which all the conditions of route location were investigated during the surveys, and on the selection of the most rational design solution. This can be reached by comparing several alternatives and using design solutions that have already been well checked in the construction and operation of roads in similar conditions.

CHAPTER 16

SURVEYING AND DESIGNING OF ROAD RECONSTRUCTION

97. Road Reconstruction

A road is reconstructed when its condition does not accord with the increased intensity and speed of traffic. In certain cases the road has to be reconstructed according to higher engineering standards; in these circumstances the width and design of the carriageway, roadbed and structures are altered and the horizontal and vertical route location is improved.

The necessity of reconstructing a road is based on observations of the actual traffic intensity on the road and on economic research data for estimating the future traffic intensities. If no economic research data are available, the traffic intensity in 10 years can be assumed to be double the present figure, and in five years to be greater by 45 per cent.

As a result of reconstruction the road class and service characteristics (design traffic speed, capacity, design loads and service life) are improved. The road reconstruction project should cover the following:

(1) improvement of the road location in plan, namely, straightening of winding stretches, increasing curve radii, improving visibility, designing of super-elevations, additional widths, and transition curves, etc;

(2) improvement of railway and highway intersections by arranging them at different levels or designing better layouts of the intersections at grade;

(3) changing the route on stretches through inhabited localities, where the road passes along narrow and winding streets with a multitude of turnings and small radii; with any very substantial increase of through traffic it may be necessary to bypass the locality;

(4) easing of steep longitudinal gradients and improvement of visibility in profile by the introduction of vertical curves;

(5) increasing of the carriageway width and, if necessary, of the overall width of the road;

(6) construction or strengthening of the road pavement structure;

(7) reconstruction of the roadbed to increase its stability, especially over swamps and in places prone to landslides, etc;

(8) rebuilding of structures to comply with increased clearances and loadings;

(9) erection of roadside buildings, service stations, motels, as well as installation of traffic signs;

(10) tree planting, etc., for aesthetic improvement.

To perform all this work during surveying and to draw up a project of road reconstruction, it is necessary to accumulate and study all data characterizing the condition of the road and structures and to draw conclusions as to the necessity of their reconstruction.

The proposed improvement of the horizontal and vertical road location, as well as the reconstruction of structures, etc., should be justified by engineering and economic calculations.

During the reconstruction survey of a road an engineer is temporarily attached to the party for investigation of the road structures and buildings. The piece rates for road reconstruction surveys differ from those applied for surveys along a new alignment and depend mainly on the length of the existing road within the limits of inhabited localities and on the land relief. When the traffic along the road is heavy the number of workers in the party is increased and the piece rates (expressed in amount of work to be done per shift) are decreased 1.25-1.65 times.

98. Engineering Surveys for Road Reconstruction

The road reconstruction project is usually carried out in two stages—project report and working drawings. For compiling the project report a detailed survey is carried out, during which it is established what work is necessary to make the reconstructed highway meet the new traffic conditions. Here account should be taken of the anticipated traffic intensity in at least 10 years.

During the preliminary work technical documents and maps are accumulated, on which are based the decisions to reconstruct certain stretches of the existing road.

The technical documentation concerning the existing road can be obtained from local road organizations. According to these documents the following is established.

(1) The general condition of the route location: traffic (intensity and composition), land relief, soil and hydrologic conditions, availability of quarries, etc.

(2) The route layout in plan: curves of minimum radius and reverse curves, watercourse and railway crossings, road intersections and junctions, route stretches in difficult topographic situations and through inhabited localities.

(3) The route layout in profile: stretches with maximum gradient, places requiring substantial earthworks, provision of adequate visibility.

(4) The types of roadbed cross-sections used in different conditions.

(5) The width, type and structure of the carriageway at individual sections and the length of the latter; data concerning the state of the pavement, measurement of pavement and subgrade depth and information concerning previous carriageway repairs.

(6) Design and measurements of road and special structures, the clearances, design loads and condition of these structures.

(7) Sources of suitable local road-building materials.

(8) The provision of drainage and the design of water collecting installations.

During the course of the field work all the accumulated material is analyzed in detail and supplemented where necessary. The main problem to be solved during the study of the material is to determine what sections of the road are still usable and where it will be necessary to relocate the road. When solving this question it should be kept in mind that the highest economy is achieved in reconstruction when use is made of the existing roadbed, structures and carriageway. If, however, the location of the road is sinuous, the grade line is at a low elevation and the carriageway is in a bad state, it is preferable to give the road a new alignment. In this case the existing road is used during the construction period for hauling materials, and later for local traffic or as a tractor track.

When relocating a road the tendency should be to reduce its length and to increase the curve radii. This will lead to a reduction of transportation and operating costs, which will amply compensate the extra cost of building the highway along a new alignment.

99. Field Work in Detailed Road Reconstruction Engineering Survey

The field work involved in the survey of a road to be reconstructed is carried out in the main according to the same rules as when surveying a new road.

On stretches where the existing road cannot be brought up to the requirements for the reconstructed road, new route alternatives are surveyed. The final solution is reached following their comparison.

On arrival at the appropriate locality the head of the party, together with the geologist and a representative of the road operating forces, examines the route. The alignment is ranged (traced) along the contemplated centre line; on metalled roads the ranging poles are placed in special supports in order not to break holes in the carriageway. On long straights it is sometimes possible to range along the shoulders parallel to the designed centre line. The apices of turning angles should be located as the intersections of the ranged centre lines of the two adjacent straights. After determining the turning angle, the bisector and tangents of the existing curve are measured,

the radius of the curve being determined according to tables for marking out curves. If the radius of the existing curve is too small, then a greater one is selected. The new curve is set out and its commencement, middle and termination points are indicated.

Sometimes the existing road, having a general straight direction, deviates slightly in plan. For this reason the centre line of the new straightened route has to be offset in relation to the existing road, and this requires widening of one side of the carriageway and roadbed, although the overall dimensions of the existing road are quite sufficient. In these cases additional small turning angles are introduced in order to make the centre lines of the old and new roads coincide.

The length of the route is measured along the centre line and all the station markers are moved to the right-hand side of the road. On the markers, in addition to the station number and plus point, the distance to the new road centre line is indicated. The following is indicated in the chainage log:

(1) the boundaries of various types of the existing road pavement, indicating the material, the state of the surfacing and the base, the width of the carriageway, the shoulders and the roadbed, the methods and state of the slope stabilization, and the roadside drain ditches;

(2) all existing structures, their situation, dimensions, material, type, design, dimensions and condition;

(3) railway crossings and highway intersections, both at grade and separated;

(4) all water collecting structures on the existing road (ditches, borrow pits, chutes, velocity-breaking steps), their state and type of protection or fortification;

(5) stretches of road situated in unfavourable conditions (landslides, talus, ground water, frost heaves, etc.);

(6) all the signs and markers of the existing road (kilometre posts, station and other markers, used for lashing the route and checking its correct measurement).

The results of carriageway inspection and pavement depth measurements are entered into a special or the chainage log.

The elevations of all stations and breaks in the profile, ditches, the carriageway on bridges, the top and runway of culverts, water levels, cross-sections under the bridge and upstream and downstream from it are levelled with two levels. When work is carried out in inhabited localities the manholes of underground structures, sewer grates, tramway rails, etc., are levelled.

The main survey work includes surveying of cross-sections, measurement of the pavement thickness and determination of its bearing strength for the following calculation of the required pavement thickness. Cross-sections are surveyed at all characteristic places

along the profile, not fewer than one for each station, and also at all places where the design of the roadbed varies, at curves with superelevations, at locations of culverts, filtering terraces, retaining walls and other structures. In mountainous regions and on steep hillsides cross-sections are levelled at each station and plus point of the route. The cross-sections are surveyed for the entire width of the road together with all its structures in order to design the drainage and locate the borrow pits.

When levelling, the elevations of the carriageway should be determined for at least three points on gravel and rubble surfacings and for five points on high-quality surfacings and pavings.

The roadbed cross-sections are drawn to a scale of 1 : 100, and those of the carriageway to the following scales: horizontally 1:100, vertically 1:20.

Examination of the pavement condition consists in its visual inspection and the boring of test holes. When inspecting the surfacing the degree of its smoothness, and the kinds of deformation and cracks are noted.

The number of cross-sections at which the surfacing is measured depends on its condition. When the state of the surfacing is satisfactory or good, the measurements are made at 3 to 5 places per kilometre; when the state is bad they are made more often. With a carriageway width up to 6 metres 3 holes are made, and with a wider one 5 holes, each 0.15 to 0.20 m in diameter. The extreme holes are drilled at a distance of 0.5 to 1.0 m from the edge of the surfacing. The holes are drilled to a depth of 5-10 cm below the sand base.

In a special log of pavement measurements the depths of the separate layers and of the entire pavement, the kind of stone or gravel materials used, the state and degree of contamination of the structural layers, and the kind of soil at the base of the pavement are registered. The depth of the pavement is measured with a special instrument to an accuracy of within 1 cm.

Investigation of the drainage consists in the surveying of borrow pit, drainage and hillside ditch cross-sections. These are levelled and the conditions of water flow studied. All existing structures such as velocity-breaking steps, flumes, chutes, evaporation basins, etc., are inspected, sketches of them are drawn and their condition is established. At the same time all data necessary for the hydraulic calculation of water drainage facilities are collected. It is desirable to observe the operation of the drainage system during and after heavy rainfall, when deficiencies in the drainage usually become apparent.

By inspection of structures and roadside buildings their condition is determined, and then drawings are made accompanied by a detailed description of the work necessary for their reconstruction.

On routes intersecting swamps, borings and soundings are carried out to establish the character, type and structure of the bog, the density of the peat beneath the embankment and adjacent to the roadbed, the profile of the bog bottom along the centre line of the route, the configuration of the embankment body and the extent of its submersion, the possibility of further embankment subsidence and dislocation, the kinds of embankment soils and the state of the roadbed.

As a result of the detailed engineering survey the party submits all the materials necessary for surveying new roads, and, in addition, a list of existing structures and their sketches, a list of work quantities connected with their repair and reconstruction, a list and charts of existing pavement thickness measurements, a list of existing signs and markers on the road, and a list and diagram of the location of existing roadside buildings.

100. Relocation of Road

The reconstructed route should be designed with the aim of reducing the road length by eliminating unnecessary undulations, straightening individual sections and by increasing the radii of separate curves. The alignment of existing roads is often most tortuous when passing through towns and cities with narrow, winding streets. Therefore when locating a new route urban areas are often bypassed (Fig. 162). The curve radii should be increased to at least the minimum value required by engineering standards not individually for each curve, but in general along the whole route. Tight curves often occur in series. Such stretches may be encountered in developing the route, as when bypassing ravines, crossing swamps, watercourses, etc. In this case the improvement of individual curves will not have the required effect, and therefore the whole stretch of the road must be relocated.

Vertical relocation of a road usually consists in increasing the elevations of the roadbed depending on the soil and hydrogeological conditions and the occurrence of snow drifts, also in decreasing severe longitudinal gradients.

Such gradients are usually found at the approaches to bridges. To decrease these gradients without altering the horizontal alignment, it is necessary to increase the height of the existing embankments and the depth of the cuttings. This method of decreasing the gradients requires complete reconstruction of the carriageway, which leads to additional expenses, partial loss of materials, the destruction of existing protection and cover to slopes, and sometimes to the reconstruction of the bridge. In this case it is most advisable to examine the possibility of entirely relocating the route with the

simultaneous improvement of horizontal and vertical road alignment.

Attention should be specially concentrated on the reconstruction of railway and road intersections. The economic expediency of grade separation depends on the cost of construction and operation of the intersection, and on the economy in transportation costs upon reducing delays at the intersection caused by interruptions in the traffic

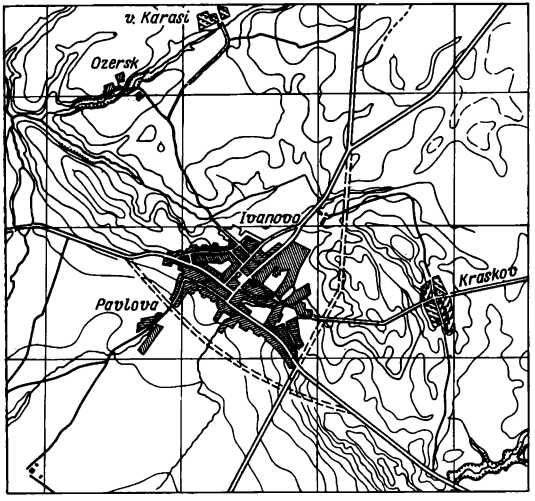


Fig. 162. Alternatives of bypassing a town upon road reconstruction

flows. The loss of time at grade intersections is due to vehicles being stopped in front of restricting traffic lights, to the reduction of speed on the approaches to them and to the necessity of accelerating the vehicle after passing the intersection. The extent of the lost time depends on the intensity of the traffic and in unfavourable cases may amount to five minutes for each vehicle, which will result in a substantial total loss of time.

A reduction of delays may sometimes be achieved by widening the intersection, i.e., where there is a two-lane carriageway four lanes may be provided at the intersection.

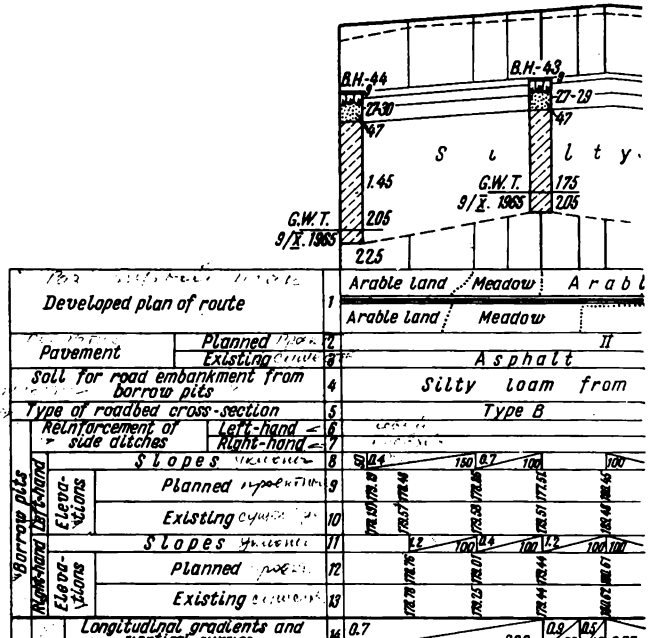
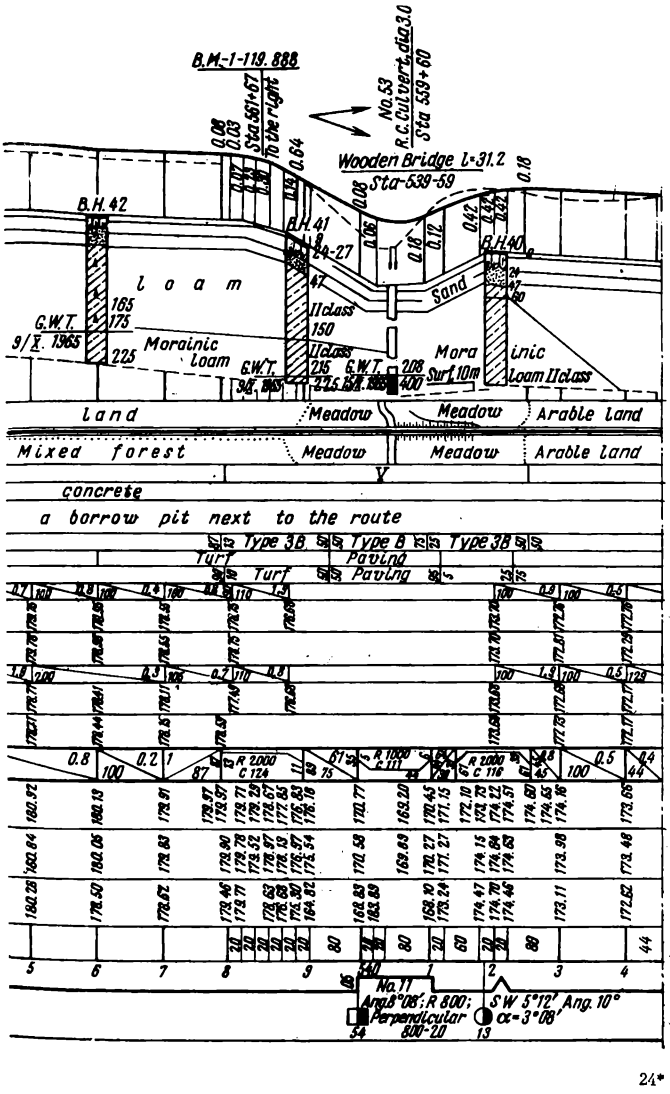


Fig. 163. Profile of road being reconstructed


B.M.-1-119.888
No.53
R.C.Culvert,dia.3.0
Sta 539+60
Sta 561+67
To the right
Wooden Bridge l=31.2
Sta-539-59
B.H.42
B.H.41
B.H.40
l o a m
Sand
II class
Morainic Loam
Morainic Loam II class
G.W.T.
9/X. 1965
Surf.10m
Meadow
Arable land
Mixed forest
concrete
a borrow pit next to the route
Type 3B
Type B
Turf
Paving
No.11
Ang.6°08'; R 800;
Perpendicular
800-2.0
SW 5°12' Ang. 10°
α=-3°08'

The profile of a road being reconstructed is plotted according to special rules. On the profile, in addition to the usual data, spaces showing the elevations and gradients of the existing road, the elevations of ditches, and the types and structures of the existing pavement are introduced. The natural ground elevation, the surface of the existing road and the grade line for road reconstruction are indicated on the drawing. At sections using the existing road the elevation differences are determined in relation to the elevations of the existing road centre line (Fig. 163).

101. Reconstruction of Road Cross-sections

For parts of a road where it is economically expedient to reconstruct the existing roadbed and carriageway, it is necessary to decide where the new road centre line will be situated in relation to the existing

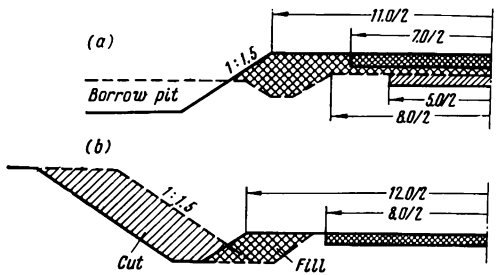


Fig. 164. Cross-section when design centre line coincides with existing centre line of road:
a—on embankment; *b*—in cutting

one. The location of the new road centre line is determined by the width of the existing roadbed. If the latter is greater than or equal to the designed width, then the new centre line is usually made to coincide with the old one. If the width of the existing roadbed is less than the designed one the following solutions may be adopted:

1. The projected centre line may be made to coincide with the centre line of the existing road. In this case it is necessary to fill in the ditches, etc., on both sides, add soil to the embankment slopes or trim back the slopes of cuttings, which may cause unequal settlement of the road and the sliding of slopes (Fig. 164).

Besides, when employing this method it is necessary to divert the traffic for the period of reconstruction, since the entire road will have to be closed to traffic. When reconstructing stretches of a road cross-

ing river flood plains and swamps the cost of diversion may be substantial.

2. The new centre line is displaced laterally in relation to the centre line of the existing road, so that widening of only one side of the roadbed is necessary.

This method is most effective when the route is laid along high embankments or in deep cuttings with stable protected slopes.

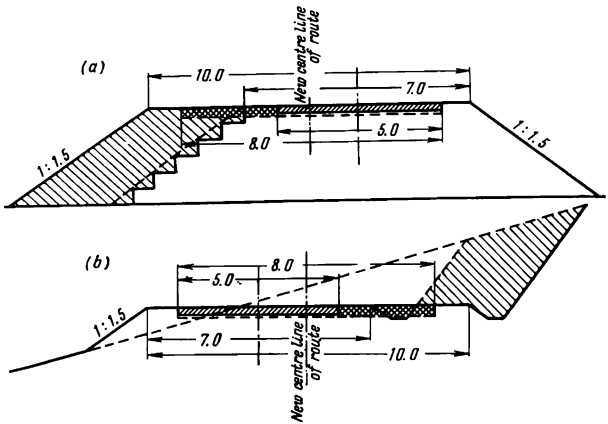


Fig. 165. Widening of roadbed to one side:
a—on embankment; *b*—on hillside

Displacement of the centre line of the reconstructed road and commencement of the road construction work first on the part to be widened and then on the existing one makes it possible to avoid the diversion of the traffic. This is most effective when reconstructing a road to first class standards, when the new centre line is deflected sufficiently for the shoulder of the existing road to coincide fully or partially with the future median.

When the existing road is sited on a hillside, the centre line is usually shifted toward the slope in order to widen the roadbed by enlarging the cutting, since the widening of an embankment on a hillside is very difficult and may require the arrangement of retaining walls. In certain cases further cutting into a hillside may lead to substantial earthworks, but nevertheless the main part of the roadbed will gain in stability (Fig. 165). Here, however, one should take into

account the general conditions of hillside stability and the possibility of occurrence of ground water and landslides.

The rational location of the new road centre line on a cross-section is established by determining the quantities and cost of work for various positions of the centre line. When the designed centre line coincides with that of the existing road, to improve cohesion between the added soil and the slopes of the embankment, benches or steps are made and the added soil is thoroughly compacted in thin layers.

When an embankment being reconstructed crosses a depression in which a reinforced concrete culvert is laid, it is more expedient to extend the culvert to one side because in this case the headwall or endwall will be preserved, and it is easier to build an extension on one side. On stretches with deep cuttings and high embankments it is also preferable to shift the new route to one side.

When designing road reconstruction special attention should be given to providing drainage for the reconstructed roadbed, and to carrying out the earthworks without disturbing the conditions of water discharge.

102. Reconstruction and Strengthening of the Pavement

Having studied the chart of pavement thickness measurement and the condition of the pavement, the *equivalent modulus of strain* is established (see Sec. 52) and the required increase of pavement thickness is computed.

Many old roads were built at very low elevations which do not correspond to local climatic and soil conditions. In such cases flooding of the roadbed by seepage water, a reduction of roadbed stability and snow drifts occur. It would be bad policy to strengthen the pavement in such conditions without elevating the roadbed. For this reason the strengthening of the pavement is based on reconstruction of the roadbed. With substantial wear of the surface and its general unsatisfactory condition an old pavement will be of no value, and the cost of its taking up will exceed the value of the recovered material. In this case the new embankment is filled in directly on the old surfacing (see Fig. 164*a*). In certain cases the height of the additional filling may be designed with the aim of using the old surfacing as the base of the new pavement. The old surfacing and the soil of its base are usually well compacted under the action of traffic and therefore it is very expedient to use them as a base.

On old stretches of the road where taking up of the old pavement when raising the roadbed is economically justified, the material of the old pavement is sorted and stacked at the roadside for subsequent reuse.

If the state of the surfacing is satisfactory and its structure after corresponding maintenance and strengthening can meet the requirements of reconstruction, the edges of the road should not be raised. The hydrogeological conditions of the roadbed are improved by deepening the side ditches and arranging flumes. Where ground water is found, side drains are laid.

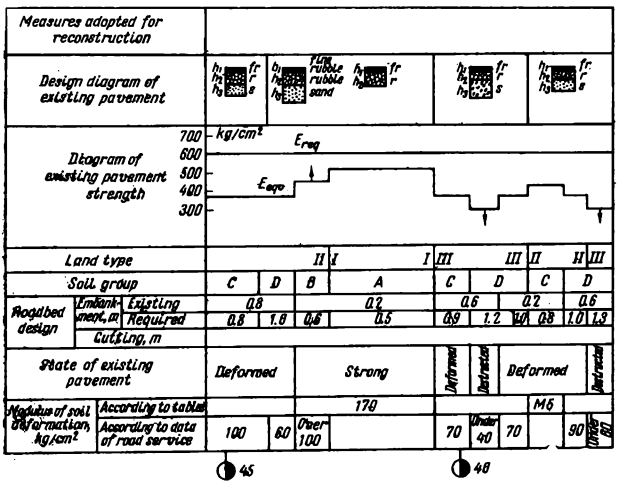


Fig. 166. Diagram of existing pavement strength

On road stretches where the height of the embankment and the state of the pavement on the whole are satisfactory, strengthening and widening of the pavement is all that will be required for the reconstruction job.

During the survey, information should be accumulated on the structure of the pavement on individual sections, and also on the composition and properties of the material used for the various pavement layers. It is most important to obtain information on road performance during periods of excessive wetting of the road and the results of pavement and roadbed tests by means of a mobile test rig during the periods of the most unfavourable weather conditions.

As a result of these investigations a diagram showing the strength of the existing pavement is drawn (Fig. 166). This is used for decid-

ing what road reconstruction work should be carried out. It is possible that on certain stretches the strength of the pavement will be sufficient, while on others its thickness will have to be increased or the pavement completely reconstructed. For convenience of work it is desirable to design stretches having a similar pavement structure of as great a length as possible, and not less than 200 m.

When the carriageway is being widened on both sides it is obvious that new strips are to be laid at each side. To improve the bond between these strips and the edge of the old carriageway, the old pavement is taken up over a width of 10-20 cm. On the widened stretches the pavement is designed to be of equal strength. It is also necessary to see that the base of the added part correctly conforms to the existing one.

Since the structure and the state of the pavement before the reconstruction may differ greatly along the road, the methods of reconstruction may have to be varied. Consequently, when compiling the profile, a special space is used to show the type of surfacing or the method of reconstruction planned for each stretch.

103. Composition of Road Reconstruction Project

The project report for the reconstruction of a road should include, in addition to the materials used for the design of new roads, the supplementary materials and documents characterizing the state of the existing road and its structures.

In the explanatory notes detailed information is given on the local climatic, soil and hydrogeological conditions. In a special section of the notes, data is included on the existing and estimated future traffic intensity, and the technical and economic justification for the need to reconstruct the road is given.

The description of the route plan gives a detailed explanation of the causes for altering it and, if necessary, the calculations justifying such relocation. The accepted alignment, coinciding or departing from the centre line of the existing road, is also substantiated. Since when reconstructing a road the utmost attention must be given to the maximum utilization of the existing pavement, the project documents must include a chart of pavement thickness measurement, data for calculating the deformation modulus and a diagram of the pavement strength.

For all road stretches which are sited in unfavourable hydrogeological and geological conditions (frost heaves, landslides, talus, etc.) plans, profiles and cross-sections, data concerning soil and geological investigations, proposed design solutions, calculations and justifications are submitted.

CHAPTER 17

COMPARISON OF ROUTE ALTERNATIVES

104. Comparison of Alternatives According to Construction and Operating Costs

During the preliminary and detailed engineering surveys, and also when working out the project report and the technical project, it is often necessary to compare route alternatives in order to select the best solution.

Alternatives may be compared in order to select the best road direction over a considerable distance (extensive alternatives) or for individual sections in complex topographic conditions (local alternatives). When comparing the alternatives it is presumed that the individual design solutions for each alternative, namely, the location of the grade lines, the type and design of the carriageway, the structures, etc., have been accepted as the most appropriate ones based on engineering and economic calculations of the individual alternatives. Only this approach can make possible the selection of the best alternative.

The alternatives can be compared by various methods, depending on the importance, length and cost of the road along each alternative, and also on the stage of designing. Thus, for instance, in the project report stage the alternatives are compared by simpler methods. When appraising alternatives one has to consider not only engineering and operational indices, but also the improvement of administrative, economical and cultural ties between localities situated in the zone served by the highway.

Approximate assessment of alternatives according to construction costs. When assessing the road alternatives the following factors characterizing the route in respect to the quantities of work, construction cost, convenience of operation and traffic safety are considered:

(1) the length of the route and its development factor;

(2) the number of turning angles (total number and number per km);

(3) the total magnitude of the turning angles $\sum_1^n \alpha$ and the average value of one angle $\alpha_{av} = \frac{\sum_1^n \alpha}{n}$, where n is the total number of turning angles;

(4) the average radius of curves, $R_{av} = \frac{\Sigma K 57.3}{\sum_{1}^{n} \alpha}$, where ΣK is the total length of the curved sections;

(5) the number of curves of minimum radius;

(6) the number of reverse loop curves;

(7) the length of sections with steep and maximum gradients;

(8) the length of sections within urban areas;

(9) the number of railway and highway intersections in one or different grades;

(10) the length of stretches with a restricted traffic speed (inhabited localities, intersections at grade, speed restrictions on tight curves, etc.);

(11) the number of major bridges and their length;

(12) the number and dimensions of special engineering structures (tunnels, snowdrift galleries, retaining walls, platforms, etc.);

(13) the number of places where traffic interruption is possible owing to mudflow streams, snow avalanches, landslides, etc.;

(14) the earthwork quantities, grouped according to the difficulty of work;

(15) the average haulage distances for the main construction materials;

(16) the requirements for main construction materials;

(17) the total amount of basic machinery and manpower required;

(18) the total engineering cost of the alternative.

The above indices enable a sufficiently accurate appraisal to be made of the alternatives being compared. However, it is very difficult to compare the various indices with each other. An alternative with a more favourable index for location in plan can at the same time be characterized by a longer average length of haul of the materials, larger quantities of work and a higher construction cost. A route alternative along a shorter alignment is often more expensive because of swamp, ravine and watercourse crossings, which were bypassed in another alternative. However, the selection of an alternative exclusively according to minimum construction cost, but of a greater length, could be the cause of a considerable increase in operating costs. Therefore, the above indices for the various alternatives should be so compared as to avoid erroneous conclusions.

Comparison of alternatives with regard to operating costs. With heavy traffic flows the operating costs for a longer alternative will increase to such an extent that in several years' time they may exceed the economy obtained by the construction of this alternative instead of a shorter and more costly one. The influence of operating costs can be illustrated by the following example. Suppose two route alternatives have a length of L_1 and L_2, where L_1 is greater than L_2.

The construction cost of the first alternative is Q_1, and that of the second is Q_2. The cost of freight transportation and road maintenance per year for the first alternative is F_1 and for the second F_2. When estimating the cost of transportation one should consider the anticipated traffic flow determined according to economic investigations. The total cost of 1 ton-kilometre of freight transportation consists of both transportation and road maintenance expenses.

The *transportation expenses* constitute by far the major portion of the total transportation cost of goods. These expenses are composed of variable costs which depend on the distance of transportation (fuel cost, lubricants, repairs, maintenance and vehicle depreciation), and of fixed costs which do not in the main depend on the volume of transportation work and which include wages and overhead expenses. The transportation expenses comprise wages—28 to 30%, fuel—14 to 18%, vehicle maintenance and repair—16 to 20%, depreciation—18 to 22% and overhead expenses—14 to 18%.

The *road maintenance expenses* consist of capital investment for construction and of outlay for the repair and maintenance of roads and structures. The latter includes the wages of road maintenance forces, winter-time maintenance, tree planting and other work involved in the operation of roads and bridges. The road maintenance part of the cost of 1 ton-kilometre of transportation is determined by dividing the total sum of the road maintenance expenses by the total ton-kilometres. The total cost of 1 ton-kilometre is $S = S_1 + S_2$. The road maintenance component S_2 on roads with high-quality heavy-duty surfacings is approximately 5 to 8% of the total cost of transportation, on medium class roads it is 10 to 12%, and on inferior roads, 15 to 20%. With an increase of traffic intensity the transportation component increases and the road maintenance component decreases.

Knowing the estimated traffic flow N and the length of the alternative L, the operating costs F for each of the alternatives can be determined

$$F = LNS \tag{224}$$

If according to the calculations $Q_2 < Q_1$ and $F_2 < F_1$, then evidently the second alternative having a shorter length L_2 should be selected. However, it often happens that $Q_2 > Q_1$ and $F_1 > F_2$, i.e., the construction costs of the shorter alternative are higher, whereas the annual operating costs for this alternative are smaller. In this case the annual economy in operating costs is $F_{ec} = F_1 - F_2$, and the increase in the construction costs of the second alternative is $q = Q_2 - Q_1$.

Therefore, if the second route alternative is selected, then the extra construction costs will be gradually compensated by the

saving in operating costs. The period during which the extra construction costs will be compensated is determined according to the formula

$$t = \frac{q}{F_{ec}} = \frac{Q_2 - Q_1}{F_1 - F_2} \tag{225}$$

If this period exceeds 12 to 15 years, then it is possible that with a small traffic intensity or a great value of q it is more profitable to select the alternative having a smaller construction cost, since it is not rational to invest in road construction excessive capital which can be returned only after a long period of time. This money could be used for the development of other branches of the national economy or for the construction of other roads. With a short period for compensation (approximately 3 to 5 years) the second alternative should be selected, which leads to greater construction costs, with a view to the fact that after this period elapses the saving in operating costs will be beneficial for the national economy.

Formula (225) can be expressed differently using the efficiency factor, or rate of return

$$\Delta = \frac{F_1 - F_2}{Q_2 - Q_1} = \frac{1}{t} \tag{226}$$

The rate of return is a reciprocal of the period of compensation. The above method of comparing alternatives according to the period of compensation gives sufficiently accurate results.

When considering alternatives it is very important to appreciate the degree of complexity of the engineering problems to be solved in each case. Thus, for instance, sections requiring the erection of major bridges, tunnels, retaining walls, etc., or which pass near landslides, mudflow streams, talus, marshlands, etc., complicate construction, increase material and labour requirements and may make it necessary to select an alternative bypassing such complex areas.

PART VI

Special Features of Road Design in Complicated Geophysical Conditions

CHAPTER 18

ROAD DESIGN IN SWAMPED REGIONS

105. Origin, Characteristics and Types of Swamps

Bogs and swamps originate and develop in areas where the soil is permanently saturated.

The distinctive indication of a bog is a peat mattress overlying the surface. A bog having a layer of peat over 0.5 m thick is called a *peat-bog*. *Swamped* districts are areas on the earth's surface on which ground and surface water accumulates, but which have no peat cover.

The main reason for swamp development is excess surface water in regions having insignificant evaporation and low temperatures, or a high ground-water table emerging to the surface. An important role in the process of swamping is played by bog vegetation—peat moss, which accumulates water. Swamps are also formed by the overgrowth of stagnant water basins or rivers having a sluggish flow. Often swamps develop following the cutting down of forests or after a forest fire, since the trees lower the ground-water table by transpiration (evaporation of moisture by their leaves). Ground water effluence (springs and wells) on gentle slopes may also be the cause of swamp formation. Swamps, therefore, originate and develop as a result of a combination of favourable conditions of topography, climate, soil and vegetation.

During their development swamps gradually pass through several stages, and their evolution is different according to the circumstances causing the formation of swamps, their origin and conditions of water inflow.

Depending on their origin, there are *floating swamps*, originated by the overgrowing of water basins and rivers, and *peat-bogs*, originating because of land swamping.

According to the situation and inflow of water the swamps can be *lowland* bogs fed by ground, lake or river water, *transition* swamps with a mixed inflow, and *upland* bogs fed by atmospheric precipitation.

The rate of peat formation depends on the swamp water conditions, climatic conditions and the type of vegetation. Decaying plants contribute continually to the layer of peat, as a result of which the nutrition of vegetation growing on the surface alters and the plant succession changes.

Water basins become overgrown from the shores towards the middle. At first, sedges, reeds and bulrushes form along the margins. Later, the basin gradually becomes overgrown and silted up by an organic silt originating from the remains of vegetation and micro-organisms (plankton) which inhabit the water, and the products of animal and vegetative life activities. Such an organic silt is called *sapropel*.

In the middle of the basin a mat of floating vegetation appears, which gradually spreads to cover the whole surface with a continuous layer, a *floating mat*. The remains of the dying vegetation gradually fill the basin and become transformed into peat. The floating mat gradually thickens until it joins the deposits on the bed of the basin. Later, grasses and shrubbery appear on the floating mat. The surface mat is often interlaced with "windows"—exposed areas of the water surface. Such openings are usually situated above springs and the deep parts of the basin. The depth of the floating mat may reach a thickness of 3-4 m, and under these circumstances may be able to sustain loads of up to 350 kg per sq m. In view of the substantial depth of floating bogs, these must be thoroughly investigated when locating a road.

Figure 167 shows the distribution of vegetation upon the overgrowth of a water basin. In such a bog sapropel covers the bottom, above it a layer of peat is formed from the deposits of decaying vegetation, and this is followed by layers of bulrushes, reeds and sedges.

In the swamping of land, the first result of the soil formation process is the appearance on the surface of a thin layer of peat favouring the accumulation of water in the soil. As the overgrowth and the depth of the peat layer increase, the surface of the bog rises, as a result of which the conditions of vegetation nutrition alter and lead to the appearance of new types of sedge, green moss, etc. Simultaneously, more favourable conditions are created for the development of shrubbery and trees, the roots of which penetrate

into the peat layer. At this stage the bog is called a *transition*, or *forest* bog.

With the constant thickening and consolidation of the lower peat layers, peat with an admixture of wood is formed in the transition stage. Later, because of unfavourable conditions, the trees gradually wither, become stunted and ultimately die out. The development of the bog enters a new phase. On the surface of the bog there now appears peat moss—sphagnum, which has a great capacity for retaining water and grows rapidly, feeding on the atmospheric moisture.

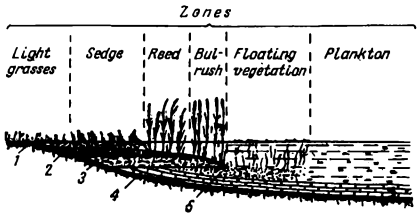


Fig. 167. Distribution of plant species in bog waters:
1—sedge peat; *2*—reed peat; *3*—bulrush peat; *4*—marl; *5*—sapropel

The lower layers of the moss die out and form sphagnum peat. The surfaces of sphagnum bogs are convex and the centres may rise 6 to 8 metres above the edges. A bog may increase in thickness at a rate of 10 to 20 cm a year. At this stage the bog is called an *upland* (oligotrophic), or *peat* bog, and is fed exclusively by atmospheric precipitation. The evolution of the bog ends with the formation of a sphagnum bog.

In such a bog peat of several types will be found, depending on the vegetation from which it was formed. The identity of the peat is determined by laboratory analysis, but in the field this is done approximately by inspection.

The peat is named in accordance with its main vegetative composition. Thus, for instance, if the peat contains 50 per cent of sedge, 33 per cent of grass, 15 per cent of reed and traces of horsetail, then it is called grass-sedge peat with reed additions.

Peat has a great capacity for absorbing moisture and can contain 10-20 times more water than its air-dry weight. Sphagnum peat has a particularly large capacity for water absorption and retention. The water absorption capacity of peat depends also on its degree of decomposition, i.e., on the degree of transformation of the vege-

tative remains into a structureless humus mass. The less the peat has decomposed, the more water it may contain. Highly decomposed peat does not contain visible remains of vegetation and, on being pressed in the hand, extrudes between the fingers. The water will not separate but the hand will become very dirty. Such peat contains up to 80-100 per cent of humus. In partially decomposed and undecomposed peats the remains of vegetation can be readily seen. When such peat is pressed in the hand, almost clear water exudes. Such peat hardly soils the hand. Highly decomposed peat closely approaches silty soils in its physical and mechanical properties. The properties of undecomposed peat depend on the strength of the vegetation remains.

Apart from organic remains, an admixture of mineral particles is contained in peat, the quantity of which determines its ash content. In lowland bogs the ash content reaches 12 to 15 per cent.

The unit weight of peat depends on the degree of its decomposition and consolidation; thus, the unit weight of undecomposed peat is 0.6 to 0.7 ton per cu m, and that of highly decomposed peat is 1.1 ton per cu m.

The water permeability of peat decreases with the degree of decomposition. Well decomposed and consolidated peat is almost impervious. Moss has a high capillary rise and a poor water yield.

Silty bogs are shallow, up to 1.5 m deep, the roots of their vegetation penetrating into the firm soil. With a further growth of the peat layer the plants may lose contact with the firm soil, and the silty bog soil becomes transformed into a peat-bog.

The design and construction of roads in swampy regions is a complex engineering problem and requires a very thorough study and survey of the bog.

From the aspect of highway design, it is most important to ascertain the bog structure in a vertical cross-section, reflecting the conditions of its formation. According to the engineering classification of bogs, developed by the Soviet scientist N. P. Kuznetsova, bogs are divided into three main types:

The first type includes bogs completely filled with peat bedded on firm ground.

The second type comprises bogs with peat of unstable consistency, with underlying organic or semi-organic silt (sapropel).

The third type includes bogs with liquid peat having a floating mat.

Having established the type of bog and knowing the thickness and properties of its separate layers, such a design of the roadbed is selected, that will ensure its stability when constructed on the bog.

106. Location of a Road in Swamped Regions

When surveying a route through swamped regions it is desirable to acquire a contour plan drawn to a large scale, on which the swamps are indicated. When surveying regions for which adequate maps are not available, good results may be obtained by the use of aerial photography.

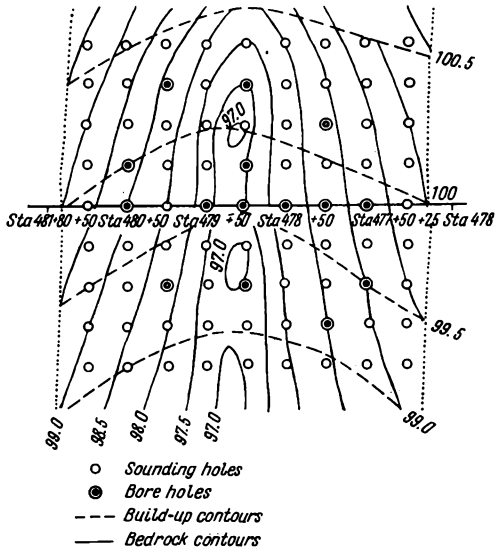


Fig. 168. Contour plan of a swamp

When locating the route the tendency should be to bypass swamps if this does not entail substantial lengthening and undulation of the route. When crossing swamps, the following rules must be observed: locate the crossing in the most narrow and shallow place; when crossing floating swamps avoid locating the route along the steep slopes of a basin with a sloping bed, as this will cause deformation of the roadbed.

In certain cases it is not possible to observe all these rules, since, for instance, the most narrow place of the swamp may coincide with

the greatest depth. In these cases the route is located after consideration of several alternatives.

The route is selected after a detailed study of maps, followed by an inspection of the contemplated alternatives on the site. A certain idea of the type of bog, the stage of its development and its possible depth can be obtained by visual inspection.

Fuller knowledge of the conditions of route location can be obtained if a contour plan of the bog surface and its firm bed is available for the width of the surveyed strip. Such a plan makes it possible to select the best route direction and should be based on the results of surveys. The plan scale and the contour intervals are set in accordance with the size of the swamp and the width of the surveyed strip. Usually its scale is 1:1,000-1:2,000 and the contour interval is 0.25-0.5 m (Fig. 168).

107. Investigation of Swamps During Route Survey

Field surveys at swamp crossings comprise topographic and geodetic works (surveying of plans, profiles and cross-sections), and the investigation of swamps by sounding, boring, as well as by selecting and studying peat samples.

Stations are laid out along all the alternative swamp crossings, and cross-sections are set out every 50 to 100 m. The width of the cross-section varies from 50 to 100 m, depending on the depth and configuration of the bog bed. The contemplated route and the cross-sections are levelled, and soundings and borings are made in order to establish the depth of the bog and to take samples of the peat.

As a result of investigating the bog the following data must be gathered: the conditions of origin and development of the bog; the types of vegetation and the botanical composition of the peat; the water circulation pattern in the bog (conditions of water inflow, etc.); the structure, thickness, density and the degree of decomposition of individual peat layers; drainage ditches; soils from the point of view of their suitability for constructing embankments over the bog, their disposition, and the conditions of excavation and haulage. The employment of permeable soils is recommended for the erection of embankments (coarse sand and gravelly soil).

The bog is sounded by means of special probes. Sounding is carried out every 25 to 50 m along the route, and at every 50 to 100 m cross-sections are set out. Along each cross-section samples are taken in three to seven places. With a shallow bog bed it is customary to drill three bore holes: one along the centre line and one on each side at a distance of 10 to 20 m from the centre line. When the bog depth exceeds 4 m soundings along the cross-sections are made in five places: one on the route centre line, two at a distance

o 10 m and two at a distance of 20 m from the centre line. On bogs with a sloping bed additional holes are drilled at a distance of 50-100 m from the route centre line. The soundings or borings in a bog penetrate into the firm soil to a minimum depth of 0.5 m, and at some of the bore holes to a depth of 2-3 m. When the difference between adjacent depths of a bog exceeds 1 m between bore holes, an intermediate sounding is made.

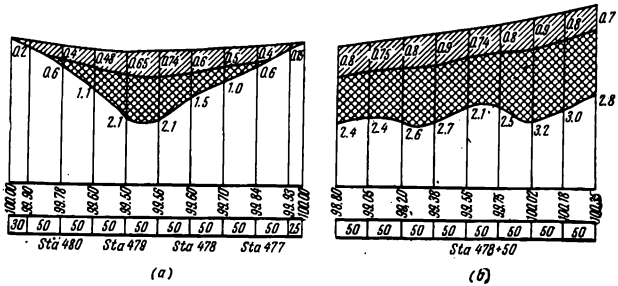


Fig. 169. Swamp sections:
a—cross-section; *b*—profile

The soundings along the route are made simultaneously with levelling, in order to avoid mistakes in determining the elevations of the bog bed.

The soundings and borings are entered in a special log, where the peat characteristics of each layer are also noted, indicating the estimated degree of decomposition, density, moisture content and botanical composition. Simultaneously, samples of peat are selected for laboratory tests. If the design of the embankment envisages the use of the bearing capacity of the peat, then for the plotting of compression curves an undisturbed peat sample is taken by means of special augers fitted with a soil sampling cylinder.

Levelling of a bog is difficult because of its soft and quaking surface. To render levelling practicable, therefore, special stakes 5 to 8 cm in diameter and 50 to 70 cm long are often driven into the bog. Notches are made on the stakes to increase their adhesion to the peat. The level can be set up on a triangular wooden support placed on the surface of the bog.

Bench marks, as a rule, are located at elevated places and on the shores of the swamps, in firm ground where the possibility of settlement or displacement is excluded.

The levelling and sounding data are used to compile profiles and cross-sections on which the depth of the bog and the types of peat layers are indicated (Fig. 169).

During the surveys the water conditions of the bog, watercourses and thalwegs are investigated and data is accumulated for the design of the structure opening.

When it is necessary to map a swamp over a large area (in order to examine the possibility of selecting new route alternatives, or to design drainage facilities) the work is done by means of a tacheometer or plane table, and the sounding and boring of swamps are carried out along the contemplated directions.

108. Design of Roadbed on Swamps

The roadbed on swamps is designed on an embankment so that the pavement bottom is raised above the surface of the swamp to a definite height, depending mainly on the climatic zone, type of surfacing and the kind of soil used for filling the embankment (see Sec. 44).

The surface of the swamp usually has an insignificant slope, and therefore the grade line is frequently designed either horizontal, or with a small longitudinal gradient.

The design of the roadbed is determined by the depth of the bog, the type and density of the peat, and also by the class of the road to be constructed.

On roads with a high-type heavy pavement on bogs up to 4 m deep and with a high-type light pavement on bogs up to 2 m deep the peat must be removed, as a rule, from under the embankment by excavation, blasting out or in other ways (Fig. 170*a*).

The stability of the roadbed depends on the strength and the bearing capacity of the peat. The roadbed slopes below the swamp surface may vary from 1:0 (vertical) to 1:0.5 (reverse slope), depending on the peat density. The effect of the weight of the embankment, pavement and vehicles is to cause the peat beneath the embankment to consolidate until the external and internal forces are in equilibrium. The designer's problem includes an estimation of the maximum settlement due to the action of all external loadings, based on the data relating to the type of peat, its density and the thickness of the separate layers. Settlement of the roadbed does not take place all at once and, if artificial compaction is not used, it may continue over several years.

To accelerate settlement the top layers of the peat are loosened or the embankment is erected by the method of gravity displacement, consisting in the initial erection of a narrower (e.g., extending over the width of the carriageway only) but higher embankment.

This increases the unit pressure at the base of the embankment and accelerates the rate of settlement. When the anticipated final settlement of the road is reached the excess earth is removed and the roadbed is widened. The rate of roadbed settlement on continuous peat foundations and comparatively dense silt depositions can be increased by the use of vertical sand drains 20 to 30 cm in diameter, located at intervals of 3 to 5 m. The vertical drains reduce the path of the water filtering out of the bed.

When constructing roads with intermediate and low types of pavement over dense peat, embankments may be erected without removal of the peat or with only partial removal thereof (Fig. 170*b*). The thickness of the remaining layer of peat, after its compaction and settlement, should not exceed one third of the embankment thickness for pavements of the intermediate type, and one half of this thickness for low-type pavements.

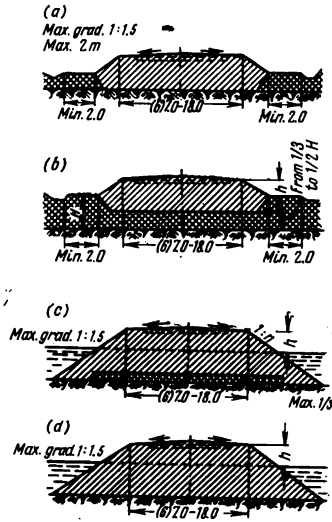


Fig. 170. Cross-sections of roadbed over swamp:

a—embankment resting on firm bed with peat completely removed; *b*—embankment with partial peat removal; *c*—embankment resting on a floating mat; *d*—embankment resting on bedrock

In all the described cases the peat must be dense and not of an extrusive type. Thus, sapropel cannot be left beneath embankments. To drain the water from the roadbed along the embankment, side ditches are excavated at intervals of 2 m (maximum) to a depth of 0.6-1.0 m and given a minimum longitudinal gradient of 0.2 to 0.3 per cent. From the ditches diversions are made for the disposal of water to depressed places. The ditch side-slopes are projected according to the peat density (usually 1:1 to 1:0.75), the width of the ditches at the bottom being 0.5 m.

To increase the bearing capacity of soft saturated soils and decrease settlement, deep compaction of these soils can be successfully achieved by means of sand piles. Investigations have shown that

the use of sand piles increases the soil strength two to three times and so reduces the settlement of the road.

On swamps with a peat mattress bedded with fluid peat or sapropel, also on floating swamps, the embankments are erected on the firm bottom of the swamp after removal of the top peat layer by blasting away or by the method of gravity displacement (Figs. 170*c* and *d*). When the bog depth is over 2 m the type of roadbed is selected after investigating the bog structure, thickness and density of individual peat layers.

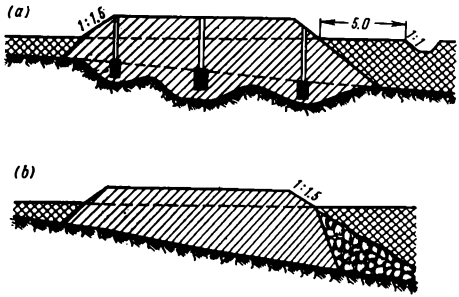


Fig. 171. Action taken to prevent embankment from shifting on a sloping swamp bed:
a—loosening of bed by blasting; *b*—construction of a stone revetment

When crossing deep bogs, wooden or reinforced concrete trestles may be erected in lieu of embankments. In individual cases when the depth of the bog is substantial, the use of precast reinforced concrete members for the trestle gives a substantial reduction in the costs and time of construction.

Roads of an inferior class can be built directly on the surface of the peat layer or on the floating mat if the latter is sufficiently securely anchored. For spreading the weight of the road over a bigger area and avoiding a concentrated pressure the road pavement is often placed over log or plank floorings. Ditches are not constructed on floating swamps, since any weakening in the tensile strength of the floating mat may lead to its destruction.

When designing an embankment resting on a swamp firm bottom having an appreciable slope it is necessary to take precautions against lateral sliding. This effect is especially dangerous when crossing floating swamps with steep banks. If the swamp bed slopes more than 10 per cent it is necessary to reduce its gradient or to arrange

benches, or to loosen it by blasting (Fig. 171*a*). At the lower side of the embankment a submerged supporting rubble revetment is constructed to prevent the embankment from sliding (Fig. 171*b*).

For erection of embankments on swamps it is good to use coarse sand or gravelly soils having a small capillary rise and not dispersing into the peat.

109. Structure Design on Swamps

When swamps are crossed, structures are erected to permit the discharge of watercourses (swamp streams), surface water or for equalizing the level of water accumulating on both sides of the embankment.

The amount of runoff and the design discharges are determined by the methods described in Sec. 37, taking into account the features of water runoff in swamps. These features are that a swamp reduces the annual runoff of a catchment area, since the evaporation from a moss-covered surface is 15-20% higher than from a water surface.

In addition, the profusion of small irregularities, mounds and vegetation causes an accumulation of a substantial quantity of moisture which is only gradually discharged. As a result the average annual water yield or discharge from swamps is appreciably reduced and the distribution of the annual runoff is more uniform.

The swamp runoff depends on the distribution of annual rainfall, the monthly average air temperature, the evaporation capacity of the area, the undergrowth and forests, and many other factors.

Bridges are to be preferred as the most desirable type of structures over swamps. Usually structures are situated at the side of the swamp where special diversion channels may be made.

When a bridge is built at the side of a swamp the work involved in the construction of the abutments and approach embankments is simplified. For draining the swamps drain ditches are designed that discharge into various facilities designed for the purpose.

When the surface slope of a swamp is insignificant it is recommended that bridges be constructed at intervals of 1 or 2 km with openings of 2 to 4 m.

When crossing a floating swamp with running water, the size of the structure is selected with a view to the depth and the velocity of the flow. With a small rate of flow, a seepage dam can be erected with the corresponding preparation of its bed. The approaches to the structure are built after the bridge and, if the firm bottom of the swamp slopes toward the bridge, the peat is removed completely within the bridge limits and replaced with mineral soil in order to avoid longitudinal sliding of the embankment and the extrusion of peat under the bridge.

CHAPTER 19

DESIGN OF ROADS IN REGIONS CUT BY RAVINES

110. Soil Erosion and Ravine Formation

By soil erosion is meant the diversified and extensive destruction and carrying away of loose soil by water and wind. A result of soil erosion may be the formation of *ravines* which, in the process of their development, pass through several consecutive stages. In the *first stage* a rain channel, or washout fan, appears on the surface of the soil. During the *second stage* the ravine apex is washed out at a considerable rate and the ravine itself begins to grow rapidly in length in a direction opposite to that of the water flow. The extension of the ravine is accompanied by collapse and caving in at its apex and by simultaneous intensive deepening of the bed. In the process of ravine erosion a flowing profile is gradually developed, which resembles the equilibrium profile of a river bed. In the *third stage* the ravine continues to deepen and widen as a result of erosion and caving in of its sides. In the *fourth stage* bed erosion and underwashing of the banks gradually cease, and the length and cross-section of the ravine become stabilized.

The slopes of a ravine, as a rule, are less steep than the angle of natural repose of the soil. Therefore, they do not cave in, and acquire a fully formed soil cover. The slopes of a ravine gradually become overgrown with grass and scrub.

A typical plan of a ravine and the character of its slopes are given in Fig. 172. The maximum gradient of the ravine slopes is observed at its apex. Nearer the mouth of the ravine, its slopes become less precipitous because of the talus, acquire a covering layer of stable soil, which subsequently becomes overgrown with vegetation.

Usually above the head of a ravine is a *hollow ab* (Fig. 173) with comparatively gentle slopes; at the ravine apex the hollow changes into a *gully cd* with steep banks exhibiting signs of bed and bank erosion; lower down the ravine widens and becomes a *valley ef* through which periodically, and sometimes continually, flows a stream having frequent changes of direction. The most intensive erosion and growth of the ravine take place in the zone of the gully, which is characterized by large longitudinal and transverse gradients. Within the limits of the valley is a zone of transit. In this

zone there is neither erosion nor silting, the eroded material being carried by the stream into the *mouth* of the ravine, to be deposited there as an *alluvial fan.*

The ravine depth and intensity of development depend on the situation of the local *base level of erosion.* The base level of erosion is the elevation at which the runoff water loses its scouring capacity. For ravines discharging into rivers, the local base level of erosion is the level of the river at the place where the ravine falls into it.

The processes of erosion are the most extensive in loess or loesslike loam. The development of erosion depends greatly on the local climatic conditions, particularly on rainfall (its distribution among the seasons and intensity). Continuous and prolonged rain or short intense storms favour the development of soil erosion. In these conditions a very great part is played by the surface vegetation which protects the soil from washout and erosion. In certain regions erosion is increased by the action of the wind, which blows away the upper layers of the soil.

The destruction of trees and grass, which protect the soil from erosion and regulate water runoff, greatly contributes to the development of erosion. Wrong techniques in agriculture (the ploughing up of hillsides, furrows running down a slope, the close cropping by cattle of slopes leading to the extinction of the grass cover) also lead to the rapid growth of a ravine network.

In some cases ravines are very extensive—many scores of metres deep and up to 15-20 km long. The largest and deepest ravines are usually of ancient origin, being related to the relief formation in the immediate post-glacial era. Ravines continue to develop until they reach strata which are not prone to erosion, or until their apices at watersheds join the apices of other ravines on the other side of the ridge.

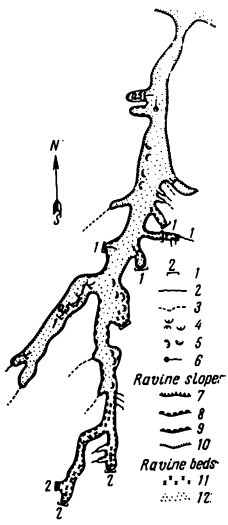


Fig. 172. Stages of ravine development:

1—scarps and falls at developing ravine apex (the figures indicate the depth in metres); *2*—gullies and scours; *3*—runoff hollows; *4*—recent slides; *5*—old slides; *6*—springs; *7*—precipitous slopes without talus; *8*—precipitous slopes with unstable talus; *9*—precipitous slopes with stable talus; *10*—steep slopes; *11*—eroded ravine bed; *12*—silted ravine bed

Ravines located in a latitudinal direction have unsymmetrical cross-sections. The slopes facing South, well heated by the sun, are precipitous and exhibit intensive erosion without cropping of

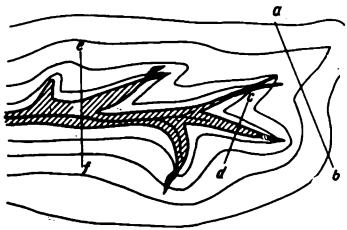


Fig. 173. Ravine plan view

bare rock. The slopes facing North and East, on the other hand, mainly have a gentle gradient and are covered with soil that is eroded to a smaller extent.

111. Road Location in a Ravine Zone

The location of a highway in a region cut by ravines is determined by the location of the towns and cities which the road is to connect and the layout of the ravine network.

According to the arrangement of the ravines relative to the highway, there may be ravines crossed by the route, approaching the road from one or both sides, and situated parallel to the road.

The route should be located with a view to the configuration of the ravine network and to the class of the road. With a highly developed ravine network, a route bypassing all the ravines will be tortuous. In these conditions highways of the highest class should be located along the shortest possible route without, of course, the excessive crossing of ravines. The increase of earthwork quantities and of the number of structures will be compensated by the reduction of the carriageway cost and, which is the most important, the reduction of transportation costs. In complex conditions various alternative routes bypassing and crossing the ravines are analyzed and the rate of return is determined for the most costly alternative.

When roads of lower classes are being designed the determining factor is the capital cost of construction. Often the most profitable alternative is the one having the minimum number of ravine crossings. When bypassing a ravine the route should be located at

a distance of 50 to 100 m above the ravine apex and its branchings. The same design provides for stabilizing of the ravine.

Figure 174 shows alternative locations, one of which follows the shortest route and crosses all the ravines, while the others partially bypass them. Bypassing of the majority of the ravines led to a substantial lengthening of the route (almost by 2.5 km) and with a heavy traffic flow this would cause a substantial increase of transport costs, apart from the higher cost of pavement construction.

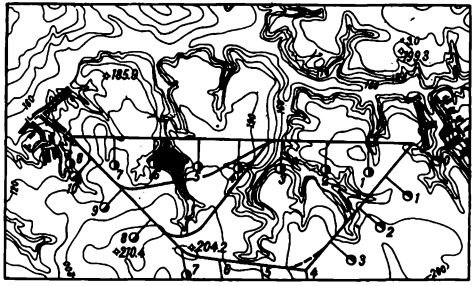


Fig. 174. Alternative highway locations in a ravine zone

If it is taken into account that when crossing a ravine the road embankment can be used, with certain modifications, as a dam for creating ponds and reservoirs, then the advantages of the alternative which crosses the ravine will grow still further.

When selecting the location of a ravine crossing, the following must be taken into consideration. The width of the ravine is less near its apex, therefore the area of a structure opening decreases the closer it is to the apex. However, location of a route in the immediate neighbourhood of the apex (in the erosion zone) is not rational, since supplementary work for the protection of the roadbed and of the structure from erosion will be necessary. It is desirable to locate the route in the stabilized part of the ravine, taking into account the stability of the ravine slopes, ground-water effluence and the possibility of landslide occurrence. When crossing wide and deep ravines it is sometimes necessary to locate the route along its slopes in order to reduce the earthworks. For crossing very deep ravines stone or reinforced concrete viaducts are constructed.

When locating roads around ravines, the case of a route following a watershed, when the ravine apices approach it on both sides, is the most complicated one. Since the route becomes very tortuous,

tight curves must be used, and simultaneously means for ravine stabilization must be designed.

Measures for halting ravine propagation are indispensable, otherwise sooner or later the constructed road will be destroyed owing to ravine development (Fig. 175).

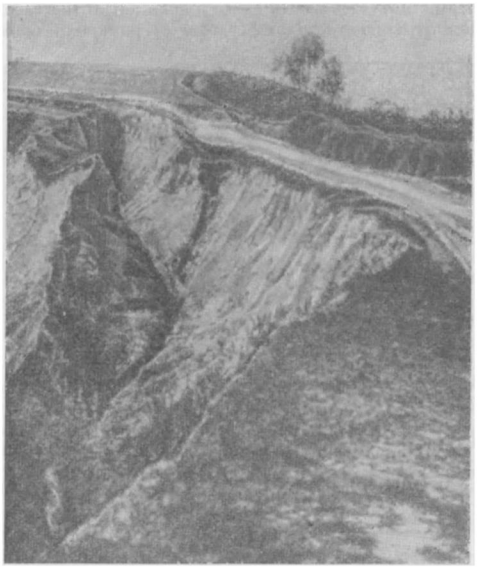


Fig. 175. Destruction of a road owing to ravine development

Special attention should be given to reinforcement of the roadbed slopes, ditches and their bottoms. Steep side ditch gradients in silty and loamy soils facilitate erosion, as a result of which an ordinary ditch may soon become a ravine.

When locating a route along a river valley, it is often necessary to cross ravines near the point of their discharge into a river, where alluvial fans are deposited. The route should not be located across the alluvial delta, since in this locality the stream-beds are constant-

ly wandering. In addition, due to the extensive deposits the openings of the structure will be clogged. In this case it is preferable to so change the location that the ravine will be crossed upstream of the alluvial fan, i.e., in the ravine zone of transition. However, if there is a great number of small ravines, the route is located across the alluvial fans in order not to increase its length. When designing structures an arrangement of approach channels and dams must be provided which will direct the stream towards the bridge opening, prevent washing out of the roadbed and clogging of the opening.

For designing a structure and preventive measures against ravine development, the ravine with all its branches must be mapped (if a detailed contour map is not available), and profiles drawn showing the ravine centre line and the centre lines of some of the branches which are located within the erosion zone.

The geological survey includes the sinking of dug holes and, if necessary, bore holes. A general picture of the soil strata can be obtained from examination of the ravine slope exposure. Bore holes are drilled in order to establish the geological cross-section at places where the occurrence of ground water or landslides is possible.

112. Ravine Stabilization

At the turn of the last century, the great Russian scientist V. V. Dokuchayev already indicated the necessity of systematic and proper prevention of erosion and ravine formation.

Methods of preventing ravine formation include erosion control, correct agricultural practice and agrosylviculture.

Since the October Revolution, soil erosion control has been widely developed in the U.S.S.R. At first, experimental ravine control stations were established for practical soil erosion control. Later, a start was made on vast operations for the radical transformation of the natural conditions of the country. In a number of regions, as a result of the beneficial influence of afforestation, soil erosion has ceased, drying winds have been eliminated and shifting sands stabilized.

Measures for counteracting the washout and growth of ravines may be *preventive* or *active*. The preventive measures include the preservation of afforestation, correct ploughing of slopes and the prohibition of cattle grazing. These measures together with active ones contribute to the rapid stabilization of ravine slopes.

The active measures vary depending on the ravine zone. Thus, in the zone above the apex it will be good practice to loosen the soil, make furrows, terrace the slopes and construct water retaining banks and ditches.

At the ravine head, catchment and water directing banks and ditches can be arranged, as well as flumes and velocity-breaking steps. In the ravine proper the bed and banks should be stabilized against erosion, and dykes and velocity-breaking steps constructed.

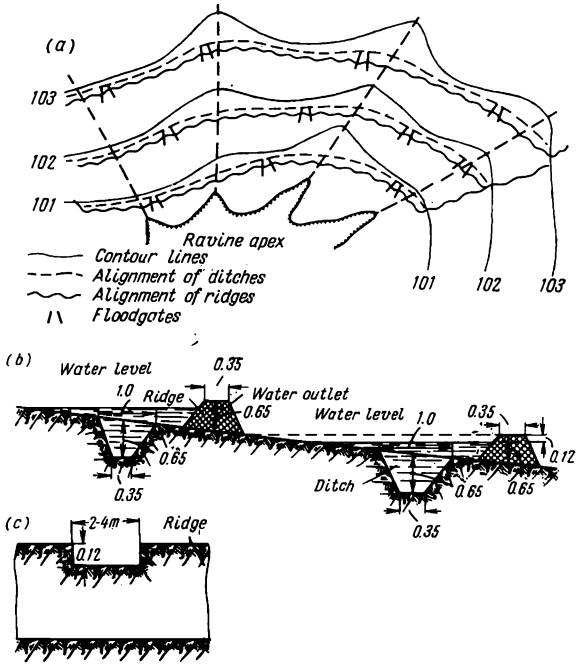


Fig. 176. Water retaining ridges and ditches:
a—schematic view; b—design; c—layout of water outlet

All these measures are carried out jointly by road building and agricultural organizations.

The active measures in the zone above the ravine apex are aimed at creating conditions for the intensive absorption of moisture by the soil. This is achieved by ploughing across the slope and producing cross-ridges of a height approximately equal to the normal

depth of snow covering. The distance between the ridges is selected from 4 to 10 m in loamy and clayey soils having a slope of from 4 to 8 per cent.

Water retaining ridges and ditches (Fig. 176) are used on steep slopes. The first ridge is usually situated at a distance of 10 to 15 m

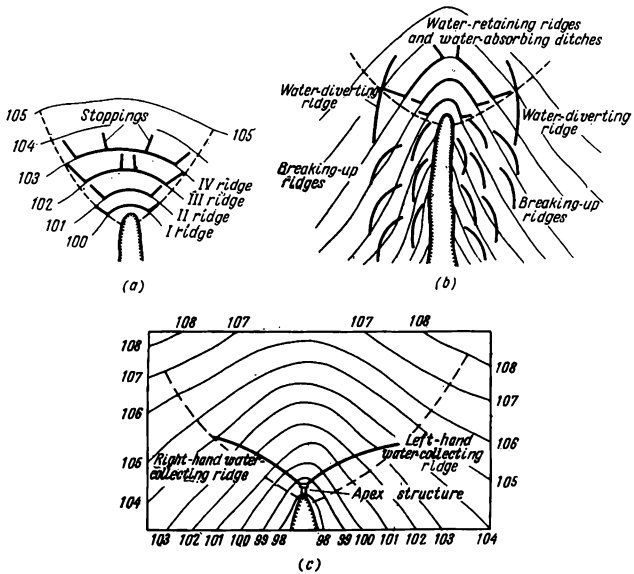


Fig. 177. Location of:
a—water reta ning ridges; b—water retaining and diverting ridges and breaking-up ridges
c—water collecting ridges next to the ravine apex

from the ravine apex, and not nearer than $2h$ or $3h$ (h being the depth of the ravine at its apex). The number of ridges and the depth of the ditches are calculated with a view to the quantity of water that gathers after prolonged rainfall (Fig. 177 *a*).

When the total runoff is great the water may be discharged into the ravine by means of special staggered ridges which break up the runoff into a series of separate streams zig-zagging down the slopes (Fig. 177*b*). With such discharge of the water part of it is absorbed

by the soil, part retained by small topographic irregularities, while the remaining water runs with a low velocity and in a thin layer down the side slopes of the ravine.

If the ravine apex is situated close to the road, the water collecting ridges and ditches must be so designed as to divert all the water to the head catch structure (Fig. 177*c*), which is a series of velocity-breaking steps, a chute or a cantilever weir with a stilling pool. In this case the ravine bed must be stabilized. The size of the ditches and the method of stabilization are determined by hydraulic calculations.

The ravine bed within the limits of the erosion zone is usually protected with brushwood which is laid on the bottom to a depth of 0.5-0.7 m and pinned down every 1.0-1.5 m with clamps held by knotty pegs. More often, brushwood steps are made 0.3-0.6 m in height. Depending on the velocity and amount of runoff the steps may be made of one or two parallel fences of brushwood wattle with a soil fill between them. Stilling platforms are protected with brushwood lining; when the falling height is more than 0.6 m, the surface is protected with turf or paving. It is good to use freshly cut willow brushwood for the brushwood lining.

A more solid type of protection may be formed of faggot wood 30 cm in diameter, laid across the ravine to build a wall 0.5 to 0.6 m high and retained by stakes driven at intervals of 30 to 40 cm to a depth of 0.7 to 1.0 m. Upstream of the faggot wall, a dyke is constructed by backfilling with clay and ramming. The ends of the dyke are cut into the side slopes of the ravine for a length of 0.5 to 1.0 m in order to avoid washing down thereof.

With the passage of time detritus is deposited between the dykes, the ravine bed rises and levels out. The distance between the dykes is so selected that the top of one dyke is approximately on the same level as the bottom of the one above it. Below the dyke, at a distance of 2-3 m, the ravine bed is stabilized with faggot wood or by paving. The design of brushwood dykes used in ravines is shown in Fig. 178.

In large ravines, and with extensive discharge at high velocities, a heavy-duty type of stabilization must be used, such as stone and concrete velocity-breaking steps, precast concrete flumes and stilling pools.

An important role in ravine fastening and in the prevention of soil erosion is played by afforestation. Trees and shrubs should be planted at the head of the ravine near its slopes and banks as a belt 20-60 m wide. Grass will develop in this belt which will stabilize the top layers of soil with its root system, retaining the moisture and, therefore, reducing the runoff into the ravine. After the completion of active measures for the prevention of ravine propagation

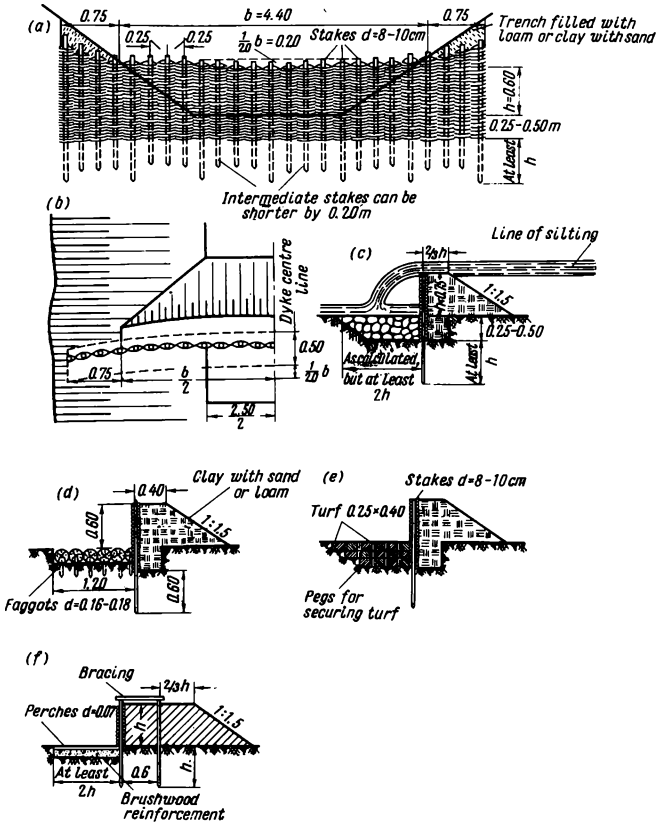


Fig. 178. Brushwood dykes in ravines with various reinforcements:
a—front view of brushwood dyke; *b*—plan view; *c*—rock fill reinforcement; *d*—faggot wood reinforcement; *e*—turf reinforcement; *f*—double-wattle dyke

and erosion, it is desirable to plant trees, etc., on the slopes and the beds of the ravines for their final stabilization.

The project of active measures against soil erosion is drawn up on the basis of the data accumulated during the surveys. In particular, it is necessary to prepare a detailed contour plan of the ravine and of the neighbouring catchment area to a scale of 1 : 2,000 to 1 : 5,000.

113. Erection of Dams at Ravine Crossings

When a highway crosses small ravines and the height of the roadbed embankment does not exceed 10 to 12 m, ponds and reservoirs can be designed. Usually the data accumulated during the road surveys are sufficient to permit the designing of a suitable dam.

When selecting the site for a ravine crossing the route should be located as far as possible at right angles to the general direction of the ravine, at the narrowest place. Dug and bore holes are excavated along the centre line of the crossing to a depth of up to 10 m at intervals of 25 to 50 m.

The inflow of storm and thaw water is calculated according to the general runoff formulas, taking into consideration the losses due to evaporation and filtration.

The dam height is selected at 0.75 to 1.0 m above the backwater level in the reservoir, the width of the crest being equal to the roadbed width. For constructing the dam local soils such as clay, loam and sandy loam are generally used. Sandy loams containing 50 to 60 per cent sand are excellent for this purpose. The upstream face of the dam is protected with riprap, a single or double layer of broken rock, or with a rock-filled wattle casing underbedded with gravel. The downstream face can be protected by turfing, seeding with grass, and less frequently by paving it. The cross-section of a dam 8 to 12 m high will differ from that of a road embankment only in having more gentle slopes, the steepness of which depends on the height of the dam and the type of the fill material. For loamy grounds and a water head of up to 6 m the slope of the upstream face may be from 1 : 2 to 1 : 2.5, while for sandy loams and sands it may be 1 : 2.5 to 1 : 3. The slope of the downstream face is usually within the range of 1 : 1.5 to 1 : 2.

Earth dam cross-sections are shown in Fig. 179. The simplest case is when the dam is filled with a homogeneous soil (Fig. 179*a*).

When there is a danger of water seepage under the dam base, an impervious cut-off is constructed (Fig. 179*b*); if the dam is erected of sand, then an impervious curtain of clay or loam is inserted as shown in Fig. 179*c*. If an impervious stratum occurs at a shallow depth below the base of the dam and there is danger of water seepage through the body of the dam, then in its body there is designed

an impervious core, which is carried down into the underlying impervious soil (Fig. 179*d*). In the case of water seepage through the dam, drainage is provided on the downstream side for carrying off the water and depressing the saturation line within the dam body.

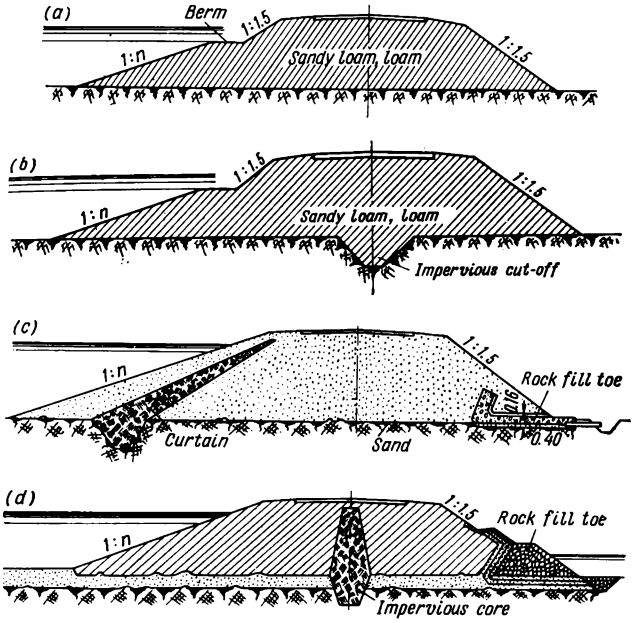


Fig. 179. Cross-section of earth dams:
a—of homogeneous fill; *b*—with an impervious cut-off; *c*—with an impervious curtain; *d*—with an impervious core

This drainage consists of rock fills in the shape of a prism arranged at the toe of the downstream slope, the draining material being of a coarser size towards the centre of the fill (inverted filter). The body of the dam is made continuous with the banks by the construction of trenches or cut-offs parallel to the bank slopes. For the discharge of flood water and partial or complete emptying of the reservoir,

outlets are made, most frequently in the form of reinforced concrete culverts with a gate in the headwall or endwall. With a sufficient quantity of water in the reservoir and a continuous water inflow, a small hydroelectric plant may be installed on the dam which could be used for agricultural needs. Calculations show that the extra earthworks needed to replace a normal embankment with a dam of equal height do not exceed 15 to 25 per cent, while the cost of

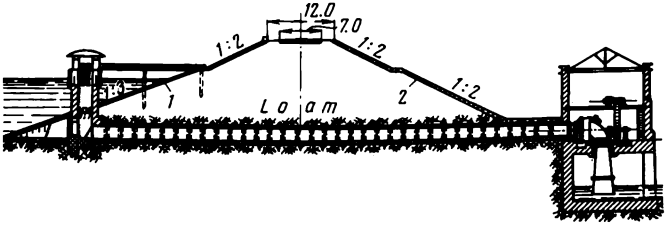


Fig. 180. Dam used as road embankment:
1—double-row paving over a layer of rubble; 2—single-row paving

the stabilization work is insignificant. At the same time when a dam is constructed, the length of the road decreases, the cost of the pavement and transportation costs are reduced and favourable conditions are created for the development of agriculture. A schematic drawing of a dam serving as an embankment is given in Fig. 180.

When designing roads even of inferior class it is not always necessary to bypass all the ravines. In certain cases it is better to locate the road along the shortest route and, if this is feasible under the local topographic conditions, to build an earth dam.

CHAPTER 20

DESIGN OF ROADS IN MOUNTAINOUS COUNTRY

114. Geophysical Properties of Mountain Roads

The survey, design and construction of mountain roads present substantial difficulties since, owing to the complex topography, the route has in most instances to be artificially increased in length. Much of the work often has to be carried out in rock, using explosives, and retaining walls and revetments are often required. Unfavourable geological conditions may be encountered such as landslides, talus, etc., and special structures have to be provided to ensure stability of the roadbed. The earthwork quantities for roads of classes III-IV may average 40 to 60 thousand cu m per km. The cost of construction in mountainous conditions also substantially grows, and for roads of classes III-IV may reach 1.3 to 2.0 million roubles per km, 20 per cent of which covers the construction of the roadbed, 10 to 12 per cent—structures and 28 to 32 per cent—the construction of the pavement.

A mountainous area is characterized by a highly broken relief with widely differing elevations, steep precipitous slopes, tortuous deep gorges, and a great number of ravines and watercourses. When designing mountain roads the route is located along valleys, hillsides and, if required, over mountain passes. Routes are located to cross mountain ranges at their most accessible points, i.e., at saddles.

Mountain topography is highly variable, as are the climatic conditions; the contrasts of the latter favour the development of exogenous processes, i.e., processes taking place in the upper strata of the earth's crust (erosion, talus, landslides and other destruction).

Mountain ranges, as a rule, are watersheds between rivers. Depressions in the ranges—*saddles*—are usually chosen for the siting of mountain-pass roads. River valleys in mountains are very tortuous. The slopes become steeper toward the river head. High stream velocities cause extensive erosion of the valley bed and banks in the upper courses and create thick alluvial deposits in the lower ones. Owing to the succession of strata encountered by the river, and also to the hardness of the rock, the erosion pattern is not uniform; for this reason the bed of a mountain river is full of waterfalls, natural dams and rapids. Deep valleys having precipitous rocky slopes and a narrow channel through which the stream flows are called *canyons*.

The location of a mountain road is determined not only by the rate of change of height with distance, but also by the configuration of the topography in plan. Of special importance in the location of a mountain-pass road is the degree of development of the valleys and their combination in plan. When the upper reaches of valleys approach the main watershed, that valley is used for locating the road which is the closest of all to the lowest and most accessible mountain pass. The selection of a suitable road alignment is greatly facilitated by the existence of a trellis pattern of valleys. In this case the grid formed by the longitudinal and transverse valleys permits the route to be located over the side ridges, thus bypassing the main range.

The conditions become very difficult for the selection of a suitable alignment when the valley pattern takes a radial form. The valleys converge in a *knot of mountains*, which is formed at the intersection of several mountain ranges.

Apart from the relief, which has a decisive influence on the selection of mountain road alignments and the design and location of structures, the climatic and geological conditions are also very important. The great elevation of mountain roads above sea level and abrupt discontinuities in the topography create different climatic conditions at various heights, which differ substantially from the climate of the adjacent lowlands.

Temperature. A well-known physical phenomenon is that the temperature of the air in mountains is lower than in valleys, the temperature drop being approximately 0.5 deg C per 100 m of rise. However, cases of a reverse distribution of temperature (inversion) are observed, when the denser cold air accumulates in closed valleys and lowlands.

The amount of solar heat received by mountain slopes varies enormously with their arrangement in relation to the cardinal points. On slopes facing South and Southwest the snow disappears rapidly; on slopes facing North and Northeast it may remain until late summer. The same conditions apply to the roadbed slopes in cuttings and on embankments. Thus, on the Pamir highroad at an elevation of 4,000 m above sea level, when the ambient temperature was 15 deg C, the soil temperature on the slope facing South was 41 deg C and on that facing North only 3 deg.

In high mountainous regions the movement of warm and cold air streams causes considerable daily temperature fluctuations. Unequal warming of slopes, the sharp temperature variations and erosion by water are all causes of slopes facing South and Southwest suffering the greatest destruction, and it is on that side that talus, alluvial fans, mudflow streams and avalanches occur. On the other hand, these slopes are more favourable for locating a road since

they are cleared from snow much quicker, and ground water and landslides occur on them less frequently. The question of whether to choose the slope facing North or South must be decided in relation to local conditions and to the geological structure of the slopes.

Rainfall. The amount of rain falling in a mountain region grows with the elevation above sea level. The maximum rainfall is in the zone of intensive cloud formation (1,500-2,500 m above sea level), after which it decreases substantially. The increase of rainfall for every 100 metres of elevation averages 40 to 60 mm.

The total quantity of rainfall depends on the geophysical situation of the mountainous region and, within the region, on its disposition in relation to humid winds. Mountain range slopes which face winds coming in from the sea receive more rainfall than the reverse side of the range. In mountain regions the wind often blows along the valleys and gorges, as a consequence of which the rainfall in the valleys is substantially higher than on the highlands and watersheds.

In summer-time very heavy storms may occur in the mountains, and about 15 to 20 per cent of the annual rainfall may be discharged in a single storm. Storms causing unusually heavy flows of water in thalwegs which are normally dry require the most careful designing of structures. Torrential streams cause river-bed erosion, the formation of mud and stone streams, and deposit extensive debris in the mouths, in the form of alluvial fans. When designing the openings of structures and the approaches to them it is necessary to give regard to the local conditions and experience gained in the operation of existing structures, since the customary methods of determining the discharge and calculating the openings are not always applicable.

Atmospheric pressure and winds. Atmospheric pressure decreases with the elevation. At low altitudes the barometric pressure drop is rapid, but the pressure declines more slowly at higher altitudes. A change in barometric pressure of 1 mm corresponds to an elevation difference called a *pressure step* (Table 35).

TABLE 35

Temperature (degrees C)	Pressure (mm Hg)			
	760	700	600	500
	Pressure step at given temperature			
—10	10.1	11.0	12.8	15.4
0	10.5	11.4	13.3	16.0
+10	10.9	11.9	13.9	16.7

The rarefaction of the air in highlands has a harmful effect on the human organism and causes the reduction of engine power. At high altitudes (3,000 to 4,000 m) there are frequently winds with velocities of 25 to 30 m/sec. At such altitudes the temperature is below freezing point for seven or eight months of the year, the annual temperature averaging 4 to 5 deg C. The depth of frost penetration may attain 1.5 to 2.0 m.

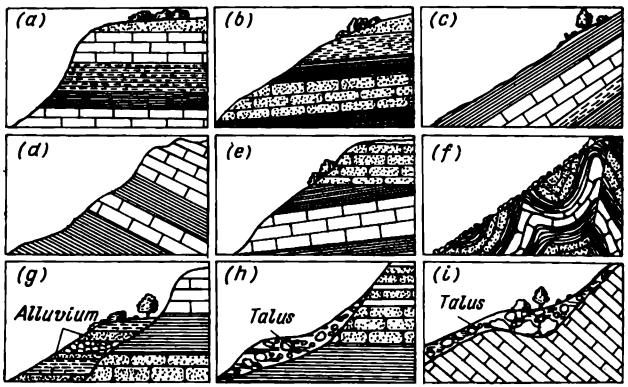


Fig. 181. Various structures of slopes:
a—horizontal strata; *b*—strata dipping towards slope; *c*—dip parallel to slope surface; *d*—strata dipping against slope; *e*—reversed fault; *f*—complex folding; *g*—unconformity; *h*—scree accumulation at bottom of slope; *i*—bedrock dipping away from slope, little scree accumulation

The appreciable difference of atmospheric pressure in valleys and on mountain passes and the sharp temperature fluctuations often cause very high winds in mountains. This leads to intensive weathering of the rock. In canyons and valleys the wind destroys gravel and broken stone pavements by blowing away the fine binding fractions. In places having a great quantity of snowfall, snow drifts and avalanches occur. For this reason it is very important for the designer to be familiar with the local climatic conditions.

Geological conditions. In mountainous regions the soil mantle is of an insignificant depth and on steep slopes bedrock outcrops to the surface and is usually covered with the products of weathering.

Stratified sedimentary rock often occurs as folds which may be concave (syncline) or convex (anticline). The inclination of the folds may vary from horizontal to almost vertical. The folds often

have various faults. Limestone or sandstone folds may be interleaved with layers of shale or clay which, when wetted, cause fracturing along their surface (fault plane), and this may result in a shear or slip fold. Figure 181 pictures various structures of slopes which occur in mountainous conditions. The degree of stability of mountain slopes depends on the type of rock, the degree of strata inclination or dip, the occurrence of clay seams, the hardness of the

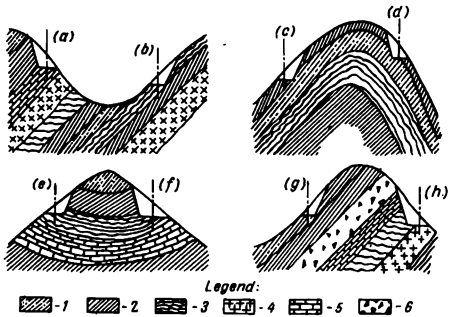


Fig. 182. Stability of roadbed [depending on strata inclination:
a, *e*, *f*, and *h*—stable location; *b*, *c*, *d*, and *g*—unstable location; *1*—sandy loam; *2*—clay and loam; *3*—clayey shale; *4*—granite; *5*—limestone; *6*—rubble

rock and the presence of ground water. When locating the route the engineer must study in detail the geology of the area and follow stable mountain slopes where no ground water, landslides and unstable folds occur. When it is necessary to locate a road along a hillside the dip of the strata should be as small as possible or, alternatively, be inclined away from it (Fig. 182).

115. Route Location in Mountains

Mountain roads tend to follow tortuous routes with great numbers of curves, the design of which is necessary to bypass obstructions, cross watercourses, and develop the route. The great number of curves and complicated topographic conditions make it necessary to use small radii (Fig. 183).

The longitudinal gradients are selected close to the maximum ones in order to reduce the earthworks and the route length. The

design traffic speed for determining the maximum gradient is established with a view to the expected traffic intensity and composition. This is particularly important for mountain roads, whose routes are located with extensive development.

Maximum longitudinal gradient. The customary method of determining the maximum longitudinal gradient according to power output characteristics is not accurate for high mountain roads, since

Fig. 183. Combination of curves on mountain roads

in these regions power output is greatly influenced by changes in atmospheric pressure, temperature and air density. A reduction in air density reduces the power of the engine and upsets its normal service conditions.

Data showing the changes in atmospheric pressure, temperature and air density in relation to the altitude are given in Table 36.

The composition of a fuel mixture is characterized by the excess air coefficient α, which is the ratio of the amount of actually supplied air to the quantity theoretically required. Automobile engines usually operate with an excess air coefficient ranging from 0.8 to 1.2. At high altitudes the air density decreases and there is a corresponding reduction in the weight of the air admitted to an engine (approximately 4 to 5 per cent per 100 m of altitude) and, with it, in engine power. As a result of the reduction in the weight of the air

TABLE 36

Altitude, m	Pressure, mm Hg	Air density, kg/m³	Air temperature, deg C	Water boiling point, deg C	Excess air coefficient α
0	760	1.225	+15	100.0	1.00
1,000	674.1	1.112	+8.5	96.6	0.89
2,000	596.2	1.007	+2.0	93.3	0.80
3,000	525.8	0.999	−4.5	90.0	0.71
4,000	452.3	0.819	−11.0	86.7	0.63
5,000	405.1	0.736	−17.5	83.3	0.56

the fuel mixture gradually becomes richer, which leads to a further decrease in engine power.

Available data show that the power of an engine decreases with the altitude as follows:

Altitude, metres	1,000	2,000	3,000	4,000	5,000
Engine power reduction, per cent	11.3	21.5	30.8	39.2	46.7

Therefore, to determine the maximum longitudinal gradient it is necessary to plot dynamic characteristics according to formulas used in the theory of automobiles.

Figure 184 shows approximate dynamic characteristics for a model GAZ-51 truck in normal operating conditions and during operation at altitudes of 2,000, 3,000 and 4,500 m. The diagram shows that the tractive effort decreases rapidly with an increase in the altitude.

However, the selection of the maximum gradient is not only an engineering problem, but also an economical one. The use of a steep longitudinal gradient makes it possible to reduce the quantity and cost of earthworks. In addition, by increasing the longitudinal gradient the length of the route is reduced, but fuel consumption grows and traffic speed is lowered. There may be cases when in spite of the route becoming shorter the time required for its negotiation does not decrease. According to the dynamic characteristic for the GAZ-51 truck, on a gradient of 4 per cent the speed is 42 km/hr, on a 6 per cent grade it is 32 km/hr and on an 8 per cent one it becomes 24 km/hr. Thus, if the gradient chosen is 4 instead of 8 per cent, the route will be doubled in length, but the speed will increase only 1.75 times, i.e., in spite of the decrease of the gradient there is no gain in time.

The increase in fuel consumption with the longitudinal gradient is the greater, the steeper the upgrade.

On steep longitudinal gradients the distribution of the load on the vehicle axles alters, which causes overloading and excessive wear of tyres and of the vehicle.

The decrease of atmospheric pressure lowers the water boiling point thus causing abnormal engine cooling conditions.

Therefore, when designing highland roads the maximum gradient for mountain passes should be less than for route stretches at comparatively low altitudes. Engines specifically constructed for operation in mountainous conditions are fitted with a special supercharger for preliminary compression of the atmospheric air before feeding it into the carburettor.

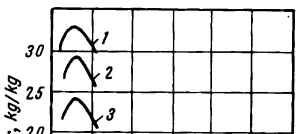


Fig. 184. Dynamic characteristics for GAZ-51 truck:

1—at sea level; 2—at 2,000 m above sea level; 3—at 3,000 m above sea level; 4—at 4,500 m above sea level

In locating mountain roads cases are distinguished when:

(1) the route alignment in plan is determined by the direction of a river valley which coincides with the selected location, or by watercourses, landslides, faults and other obstructions which have to be bypassed; such a route is called a constricted or "*forced*" *location in plan*;

(2) the natural slope of the land exceeds the maximum gradient and makes it necessary to develop the route; such cases occur when designing routes over a pass; such a route is termed "*forced*" *in profile*;

(3) development of the route along a hillside is limited by ravines, watercourses and landslides, hence the route can be located in only one direction; in such design conditions the route is called "*forced*" *in plan and in profile*. This case is the most complicated one.

The great number of tight curves on hillsides, combined with appreciable longitudinal gradients, requires careful attention to the provision of adequate visibility, adopting special measures for this purpose.

116. Route Location in a Valley

The location of a route along a river valley is the most frequent case of mountain road alignment, owing to the advantages obtained in running the road up the valley at a comparatively gentle gradient, to the proximity to inhabited localities situated next to the watercourse, to the convenience of road operation and water supply.

However, a valley run may involve numerous horizontal curves, the construction of large bridges over tributaries, and stretches below steeply sloping hillsides, which in some places may be insufficiently stable. Besides, extensive earthworks are required for building a valley run and it is necessary to construct special retaining and protection walls when locating a route on a slope, together with protective structures for safeguarding against avalanches.

A route is located along a river valley according to the following considerations.

The height of the roadbed above and its distance from the maximum water level in the river should be so selected that there will be no possibility of erosion or seepage at high water.

In narrow, constricted valleys with precipitous rocky slopes the roadbed frequently has to be placed very near to the watercourse. In such cases the embankment slope facing the river should be thoroughly stabilized.

When locating the route major attention must be given to the geological and hydrogeological structure of the valley slopes. In unfavourable geological conditions it may be essential to carry the road over a bridge to the other side of the valley in order to avoid crossing unstable ground or a tributary where mudflows may occur. Hence, the route alignment is governed by the information obtained during preliminary geological and hydrogeological surveys of local conditions.

To reduce the earthworks, particularly rock excavation, the route should be located along hillsides with the lowest practical gradients and following the main undulations of the valley.

When crossing watercourses, several route alternatives may be investigated. In Fig. 185 the *first alternative* consists in crossing the watercourse near its junction with the river, over the alluvial fan. When selecting such a solution it should be remembered that within the limits of the alluvial fan frequent changes in the position of the stream beds are possible. In these circumstances the main watercourse periodically alters its direction and flows along one or another arm of the delta, and this may cause accumulation of deposits immediately above the bridge, blocking off its opening. The building of a bridge over only one channel with the arrangement of approaches and the building of solid embankments over other beds cannot be considered as a correct solution, since cases have been experienced where the stream has eroded the embankment and flowed along a new direction, leaving the bridge high and dry. The installation of deflecting and protecting dykes with heavy fortifications also cannot afford protection in all cases from the action of the stream when it flows at a very high velocity. In the majority of cases it is

preferable to cross the entire flood plain with one continuous viaduct or bridge, on the assumption that all the bridge openings will not function simultaneously. Therefore, construction of the road according to the first alternative is connected with a substantial outlay

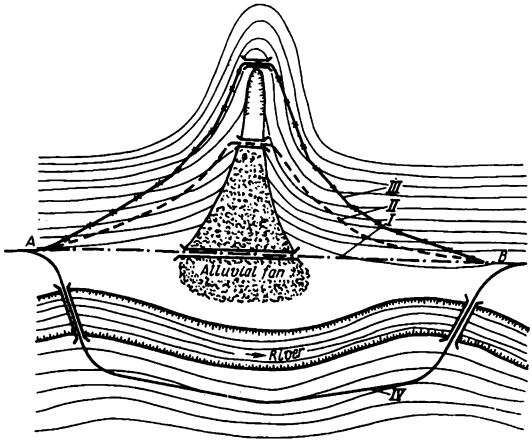


Fig. 185. River crossing alternatives

for erection of the bridge and the various structures required for its protection.

The *second alternative* is located above the alluvial fan, in the transit zone of the river, where there are no deposits. The dimensions of the structure for this alternative are less than for the first one, but are still substantial, while the length of the route is considerably increased.

The *third alternative* involves the deep penetration of the route into a side valley in order to reduce the size of the structure and to reduce the earthworks on its approaches. However, this alternative is substantially longer, and the earthworks for the construction of the carriageway are greater. If in the proximity of the watercourse mudflows are anticipated, it will be necessary to design a still deeper penetration of the route into one of the side valleys branching out from the main one, in order to cross it over a still narrower watercourse.

Since such a solution is coupled with a substantial lengthening of the route, in certain cases when the width of the river is not great a *fourth alternative* is possible. In this alternative the road crosses over to the other bank of the valley, avoiding crossing of the bed at the places where mudflows occur, and then returns back to the original bank. In this alternative two bridges have to be constructed. Its advantages, therefore, depend upon the size of the bridges and their cost. The final selection from all the alternatives is made as the result of engineering and economical comparison.

The longitudinal slope of mountain rivers is usually less than the maximum gradients acceptable for mountain roads, therefore, route location along a valley may cause difficulties only in plan view. However, in the upper reaches of watercourses, or in the case when a canyon is used for a road over a mountain pass, the natural longitudinal gradients may substantially exceed the maximum ones. In such cases the route has to be lengthened and developed, using side valleys for this purpose or aligning it on zig-zags across the valley slope (Fig. 186).

When locating a route through a narrow canyon, it may be necessary to blast out the route across the rock face and to construct tunnels and semi-tunnels (Fig. 187).

When laying a route across valley slopes a great number of cross-sections have to be taken, on which later, when the grade elevations are known, the roadbed sections can be plotted, and the corresponding earthwork quantities determined. The cross-sections should be taken at all the characteristic breaks of hillside contour and at plus points where the profile of the route changes. The length of the surveyed cross-section should be sufficient for locating the design roadbed.

On precipitous mountain slopes the survey is generally carried out by means of a theodolite, using a tacheometric technique to locate the characteristic points of the land slope. A theodolite survey is time-consuming and may not give a detailed and precise characteristic of the slope. Under such conditions the cross-sections can be advantageously surveyed with the aid of a camera. The relation between objects on a photographic plate is given with the highest precision when these are situated at the same level as the lens and in the centre of the photograph. If the photograph includes an object whose dimensions are known, then it is easy to determine the horizontal and vertical dimensions of all the other objects which are situated in the same plane. Thus, if on a photograph of a mountainous slope at a determined point a staff is photographed, then all the horizontal and vertical distances of objects situated in the same plane can be determined and a cross-section plotted.

Figure 188 shows a photograph of a mountainous region with a scale plotted in accordance with the size of the staff, and also

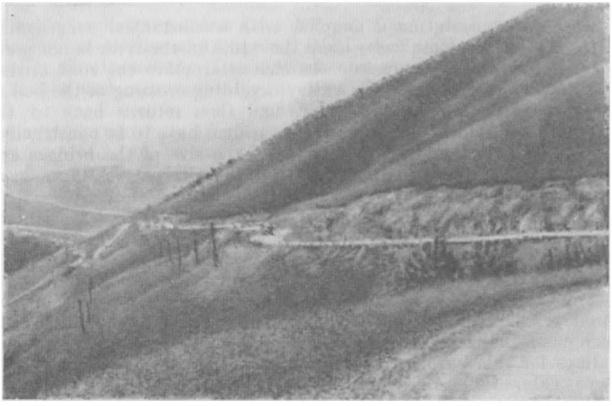


Fig. 186. Road located on hillside

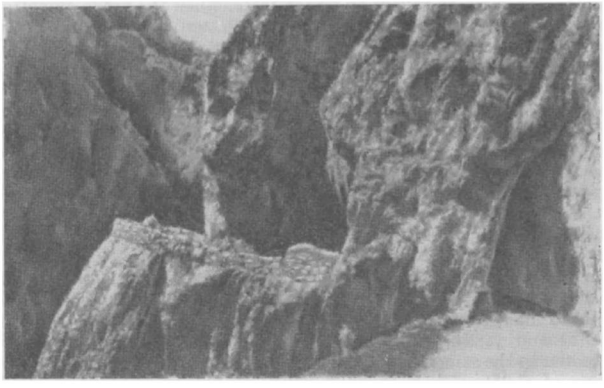


Fig. 187. Semi tunnel on a mountain road

depicts the drawing of a cross-section that accords to the data of this photograph. The precision of the photographic method checked by an ordinary survey is amply sufficient for design purposes. At the same time, this method saves much field time. According to the

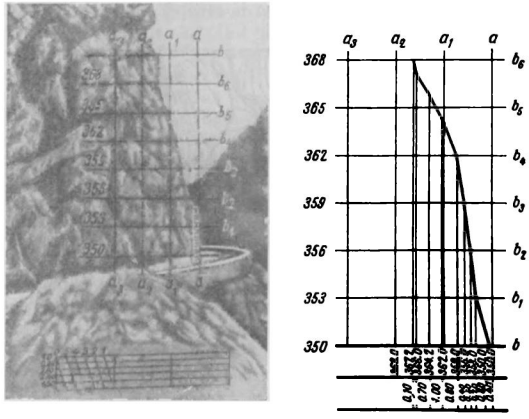


Fig. 188. Photographic survey of cross-sections: photograph of a slope (left); cross-section (right)

recorded photographic data an accurate assessment of the geology of the slope, the rock formation, the extent of denudation, etc., may be obtained, all of which are of considerable value to the designer.

117. Roads Through Mountain Passes

Mountain pass roads are characterized by their very steep longitudinal gradients, their numerous curves, their hairpin bends and the extensive quantity of rockwork involved in their construction. Additional to these factors, mountain pass roads require the construction of special engineering installations (retaining walls, snow fences and, in certain cases, tunnels, etc.).

For the preliminary selection of the pass to be used, available topographical maps should be carefully studied, a reconnaissance survey made, using barometric levelling to establish elevations, pack animal and pedestrian tracks should be investigated and, in totally unexplored regions, air photography used. Valuable infor-

mation for the selection of a crossing may be obtained by means of a survey from a helicopter. The pass to be chosen is the one which has the least elevation, which is close to the given route direction and which has the most convenient approaches for route development.

The main feature of a mountain pass is the necessity for lengthening of the route. If the land slope is steeper than the established

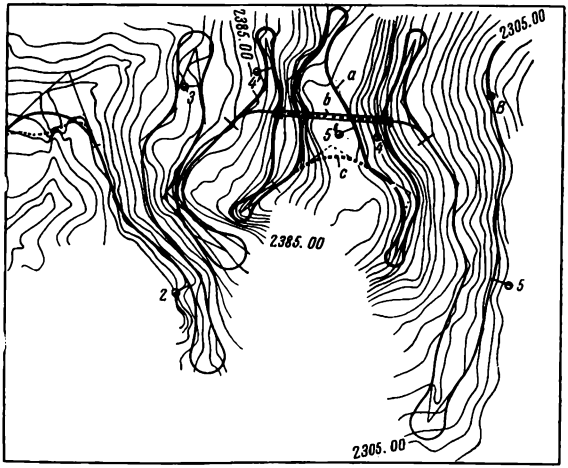


Fig. 189. Alternatives of mountain pass route:
a—developed route; *b*—with tunnel; *c*—with deep cutting

maximum gradient, the route cannot be laid along the shortest direction. In this case it has to be artificially developed. The route is developed using as a guide not the maximum permissible gradient, but a slightly smaller gradient, which may be called the *ruling gradient*. The ruling gradient is usually 0.5-1.0 per cent less than the maximum one, leaving a certain margin for any required reduction in the length of the route in order to avoid too many horizontal curves. It should also be borne in mind that the maximum gradients must be reduced on tight curves in order to improve traffic conditions.

If data describing the geological structure of the locality are available, the gentler and most stable slopes are selected for devel-

opment of the route. After this, the contemplated route is transferred to the ground and is finally corrected during this process.

In open country a theodolite is used for the development of a route. A level equipped with a suitable gradiometer screw can also be used. The instrument is set up at the origin of the route with the telescope at an angle corresponding to the accepted ruling grade. The observer sends out the staff bearer in the direction of the future route, to a distance of 50 to 100 m. The level of the instrument is marked on the staff. By moving the staff to the right or left, the observer locates the staff at a point where the intersection of the cross-hairs coincides with the mark on the staff. From the same position several staffs can be set. If visibility is obstructed, the theodolite can be transported to the place of a staff, and the staffs moved in the direction the route is to follow. After development of the route separate sections are straightened and the turning angles are established. In difficult places a main survey line is marked out, which roughly coincides with the route, and a strip 100-150 m wide is surveyed with the theodolite. According to the results of this survey a contour map is drawn, the route is plotted on it in the office, and then final correction is carried out in the field. If air photographs are available the problem of route design becomes much simpler. Other methods of route location with development of the route are also known.

As can be seen from the above, the selection of the pass is very important. It is obvious that lowering of the pass height by h metres makes it possible to reduce the length of the approaches on both sides by the amount

$$L = \frac{200\,h}{i}$$

where i is the longitudinal gradient of the road in per cent.

An example of mountain pass route location is shown in Fig. 189.

118. Tunnels

When crossing high and steep mountain ranges, where the requirements for route development will lead to a substantial increase in its length, it may be better to locate the route through a tunnel. The design of a tunnel will reduce the length of the route, the number of curves, will eliminate the danger of avalanches and rock falls, and will cut road operating costs.

The drawbacks of the tunnel alternative include the high capital cost and the complexity of construction. When laying mountain pass roads involving tunnelling, several alternatives of the tunnel must be compared.

The transition from a cutting to a tunnel is selected according to geological data, economic considerations and the convenience of executing the work. It is usually considered that the transition from a cutting to a tunnel should take place where the construction and operating cost for an open cut is equal to that of a tunnel of the same length. If the "throw-out" blasting method is used for the excavation of cuttings, the depth of the cuttings at which it is expedient to construct a tunnel is in the range of 20 to 35 m, depending on the cross-section and length of the tunnel, on the geological

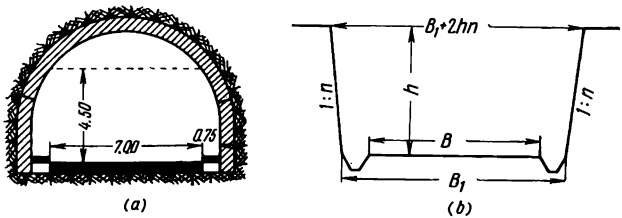


Fig. 190. Cross-sections:
a—of tunnel; b—of cutting

and hydrogeological conditions and on the construction costs. The width of the carriageway in a tunnel is usually fixed at 6 to 7 m, with additional sidewalks 0.75 m wide at one or both sides. The height of the tunnel is chosen to correspond to the clearance of bottom-road (through) bridges, allowing for the construction of ventilating chambers if necessary. The cost of construction of a cutting and a tunnel is calculated approximately as follows.

Suppose that the cost of excavating one cu m of tunnel complete with lining is m times higher than that of excavating one cu m of cutting at the approach to the tunnel. If the area of the tunnel is A_t, then with a view to the cost factor m, the reduced area of the tunnel, i.e., the area of a cutting with the same construction cost, will be

$$A_{red} = mA_t \tag{227}$$

At the approach to the tunnel the cross-section of the cutting has an area A_c equal to (Fig. 190)

$$A_c = B_1h + nh^2 \tag{228}$$

The width B_1 is equal to the width of the roadbed plus that of two trench drains at their top. Equating the area of the cutting to the reduced area of the tunnel yields a quadratic equation which gives

the depth of the cutting at which the cost of a metre of cutting and a metre of tunnel is equal, i.e.,

$$h = \frac{-B_1 + \sqrt{B_1^2 + 4nA_{red}}}{2n} \tag{229}$$

When a cutting is excavated by blasting, the actual cross-section will differ greatly from the designed one because the side slopes will be more gentle and the width of the cutting at the top will be greater. Consequently, the alternative involving the building of a tunnel will become more advantageous at a somewhat lesser depth of the cutting than obtained from formula (229).

Tunnels are designed in plan and profile according to the standards for open stretches of highways, with a view to the following additional requirements.

The minimum radius of a horizontal curve in a tunnel is 200 m, and only in exceptionally complicated conditions can it be reduced to 100 m. The minimum longitudinal gradient in tunnels should be 3 per cent, although in special cases it may be reduced to 2 per cent. When a tunnel exceeds 300 m in length, the maximum longitudinal gradient should not be more than 4 per cent. With tunnels less than 300 m long, a straight fall gradient should be adopted, but two outward-falling gradients are allowed for a tunnel over 300 m long.

The cross-section and clearances of highway tunnels depend on the type of tunnel (mountain, urban) and its equipment, and on its capacity, i.e., the number of traffic lanes, traffic intensity, the provision of sidewalks and cycle tracks.

When designing tunnels on curves with a radius of 300 m and less, the carriageway should be widened as shown in Table 37.

TABLE 37

Number of traffic lanes	Curve radii, m			
	100	150	200	300
	Additional road width, m			
Single	0.5	0.4	0.3	0.2
Double	1.0	0.7	0.5	0.3

In tunnels over 300 m long turnout chambers are designed 4 m wide, 6 m long and 2.8 m high which are located on alternate sides at intervals of 300 m. In addition, bays spaced 100 m apart and 2 m wide, 2 m long and 2.8 m high are provided on one side of the tunnel. In tunnels with a circular section, instead of these bays a footpath ledge may be constructed at least 0.6 m wide at a maximum level

of 1 m above the carriageway. In single-lane tunnels the ledges are situated on one side, and in two-lane tunnels on both sides.

In tunnels over 400 m long forced ventilation must be installed, whereas in tunnels from 150 to 400 m long such ventilation is necessary only in special cases. The velocity of the air stream in ventilated tunnels should not exceed 5 m/sec.

In tunnels in country regions with a length over 300 m on straights and 150 m on curves, and in all urban tunnels irrespective of their length, artificial lighting must be installed. The minimum tunnel illumination at carriageway level should be as follows: at night 10 lx, in the daytime 120 lx at the portals and 20 lx in the middle of the tunnel; during tunnel inspection and overhaul it should be 25 lx.

In seismic regions tunnels are designed and constructed to comply with the relevant standards and rules.

Tunnels in regions having a cold climate should be fitted with special devices to prevent the formation of ice.

The performance of vehicles in tunnels has special features, which should be considered in the speed-time-distance calculations:

1. The coefficient of adhesion is higher than on open roads and on moist surfacings, and ranges from 0.4 to 0.5. The reason for this is that the carriageway in tunnels is protected against rainfall, except for the short stretches at the entrance and exit where the vehicles may carry in moisture and mud.

2. The air resistance is higher owing to the additional compression of the air streams in the limited clearances between the moving vehicles and the tunnel walls, and owing to the turbulence of the air streams. The resistance of the air increases appreciably in tunnels over 500 m long and continues to increase with a growth of the length. When the length of a tunnel is approximately 1 km, the air resistance for trucks increases by about 40 per cent, and for passenger cars by about 10 per cent in relation to the figure for an open stretch.

The designs of road tunnels, and the methods of computing and constructing them are covered by special textbooks.

119. Design of Reverse Loop Curves

When developing a route in mountainous country, it is frequently necessary to insert sharp turning angles, within whose limits it is very difficult, and sometimes even impossible, to lay out curves following normal geometric standards of design.

When inscribing a curve inside a turning angle the length of the route will be substantially reduced, which will result in steep longitudinal gradients. Easing out of the latter will entail excessive

and expensive earthworks. In such circumstances it is preferable to round off the route not by inscribing, but by circumscribing the curve around the turning point. Such curves are called *reverse loop* or *hairpin curves* (Fig. 191).

Figure 191*a* shows a reverse loop consisting of a main curve C, reverse curves C_r and tangents m. The acute angle of the loop is α. The loop main curve, with a radius R, has a total length C and subtends an angle γ at the centre. Points A and B are located at the

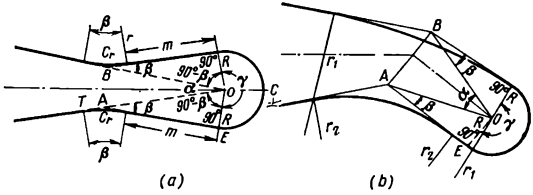


Fig. 191. Reverse loops:
a—of first type; *b*—of second type

apices of the reverse or auxiliary curves. Between the ends of the reverse curves and the main curve of the loop tangents must be introduced which are used to locate the transitions to the superelevations, the easement curves and the transition to the greater width on the curves.

The design of a reverse loop comprises the setting out of the separate elements and checking in the field the possibility of locating the roadbed together with the ditches and slopes.

A reverse loop is located on a hillside having the minimum slope and the maximum stability. It must also be safe from the point of view of landslides and ground water. The selection of a gently sloping hillside leads to an appreciable reduction in the quantity of construction work.

For the design of reverse loop elements the radii of the main and reverse curves (R and r) and the length of the tangent (m) are initially selected. First the turning angles of the reverse curves are determined at points A and B by the following method. The length of the tangent of a reverse curve is related to the turning angle of the curve according to the formula

$$T = r \tan \frac{\beta}{2} \tag{230}$$

where T = length of the tangent, m
r = reverse curve radius, m
β = deflection angle, degrees.

The distance from the apex of the reverse curve angle to the commencement of the main curve is $AE = BF = T + m$.

From the triangle AOE or BOF it will be found that

$$\tan\beta = \frac{R}{T+m} = \frac{R}{r\tan\frac{\beta}{2}+m} \tag{231}$$

where R is the radius of the main curve in metres.

From trigonometry it is known that

$$\tan\beta = \frac{2\tan\frac{\beta}{2}}{1-\tan^2\frac{\beta}{2}} \tag{232}$$

Substitution of this expression for β in the preceding formula and solution for $\tan\frac{\beta}{2}$ yields

$$\tan\frac{\beta}{2} = \frac{-m+\sqrt{m^2-R(2r+R)}}{R+2r} \tag{233}$$

whence the angle β can be determined.

The distance from the apex of the reverse curve angle to the centre of the main curve is determined by the expression

$$AO = OB = \frac{T+m}{\cos\beta} = \frac{R}{\sin\beta} \tag{234}$$

The central angle γ corresponding to the main curve of the reverse loop is equal to

$$\gamma = 360 - 2(90-\beta) - \alpha = 180 + 2\beta - \alpha$$

and the length of the main curve is

$$C = \frac{\pi R\gamma}{180} \tag{235}$$

Hence the total length of the reverse loop is

$$S = 2(C_r + m) + C \tag{236}$$

where C_r is the length of the reverse curve, m.

Having obtained these data, the reverse loop can be traced on a contour plan, or set out on the ground.

The calculation given above is for a symmetrical reverse loop, having reverse curves with equal angles and of equal radii. If, owing to land conditions, these curves should differ, the loop is designed by the same method, separately for each reverse curve.

The above loops, which have reverse curves situated with their convexities in opposite directions, are called *reverse loops of the first type.*

In *loops of the second type*, which may also be either symmetrical or asymmetrical, the reverse curves both have their convexities facing towards the same side (Fig. 191*b*).

The contour of the reverse loop depends exclusively on the configuration and geological structure of the hillside. For this reason it is selected with the aim of providing the most stable roadbed, the best conditions for vehicles to traverse it and the minimum possible quantities of construction work. The most advantageous location and form of a reverse loop may have to be established by the comparison of several alternatives.

For the calculation of loop elements it is possible to use tables which facilitate the tracing of reverse loops with tangents or transition curves. The introduction of transition curves will improve conditions for vehicles. The longer the transition curve, the less will be the build-up of radial acceleration. A detailed contour plan and patterns of curves of various radii to a correct scale will greatly facilitate the tracing of several alternatives of various types of reverse loops. For comparing the alternatives, profiles and cross-sections are plotted, the grade line is drawn and the quantities of work are determined, taking into consideration the geological structure of the land.

On mountain-pass sections of mountain roads, when the relief of the country and the hillside location of the road require a sharp change in the direction of the route, use may be made of reverse loops. The distance between the end of the auxiliary curve of one reverse loop and the beginning of the auxiliary curve of the next loop should be as great as possible, and at any rate it must be at least 400 m for class II and III roads, 300 m for class IV roads and 200 m for roads of class V.

The geometrical elements of a reverse loop are selected with a view to the established design speed and the traffic intensity. Design speeds of 15 to 20 km/hr are tolerated on loops only in especially restricted conditions on roads of classes IV and V. The recommended geometrical elements of reverse loops are given in Table 38.

The design of the elements of a reverse loop must be substantiated by engineering and economic calculations.

The branches of the road in a reverse loop curve are situated one above the other (Fig. 192). With small radii of the main curve, and if the apices of the reverse curves are near to each other, cases may occur when the roadbeds of the two branches cannot be accommodated at the place of the greatest approach (the neck of the loop). In such limiting conditions a retaining wall must be constructed to

TABLE 38

Elements	Magnitude of reverse loop element at various design speeds, km/hr		
	30	25	20
Minimum radius of main curve, m	30	20	15
Degree of super-elevation, %	6.0	6.0	6.0
Length of transition curve, m	30	25	20
Widening of carriageway, m	2.0	2.5	3.0
Maximum longitudinal gradient within loop, %	3.0	3.5	4.0

support the upper branch. This wall also serves as a side one for the lower branch (Fig. 193). If the roadbed cannot be accommodated

Fig. 192. Reverse loop on a mountain road

even with the use of a retaining wall, then the location of the reverse curve apices has to be altered.

However, vehicle speeds have to be reduced over reverse loops, and their construction substantially increases the capital cost of the

road because of the extensive earthworks and the necessity for constructing retaining walls. When designing mountain roads several route alternatives are usually investigated, preference being given to the one having the least number of reverse loops.

In certain cases it is expedient to develop a route by means other than the design of loops. For example, Fig. 194 shows a contour plan

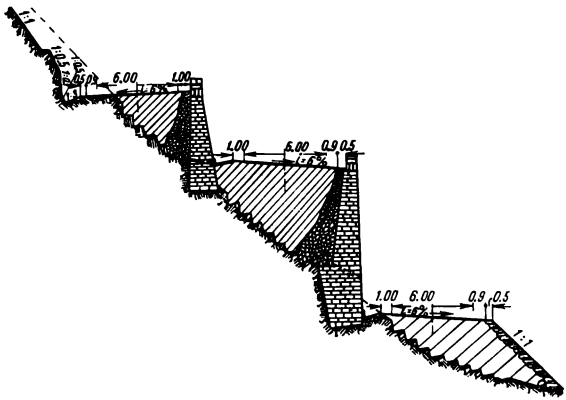


Fig. 193. Cross-section through three road branches

of an area over which a route is to be developed. When the route was designed with reverse loops, it included curves of small radius with inadequate sight distance. With spiral development the upper branch of the route is carried over the lower one on a viaduct or, if the spur is narrow and high, the lower branch is located through a short tunnel. Such a design enables the curve radii to be increased and, seeing that all the curves turn in the same direction, the traffic speed increases, the construction of the super-elevation is simplified and the sight distance is substantially improved. In addition, owing to the reduction of earthwork and stabilization work quantities, the overall construction cost will be reduced.

120. Mountain Road Cross-section

The cross-section of a road in hilly or mountainous terrain is determined by the natural slope of the site, the slope of the roadbed sides depending on the stability and density of the hillside soil.

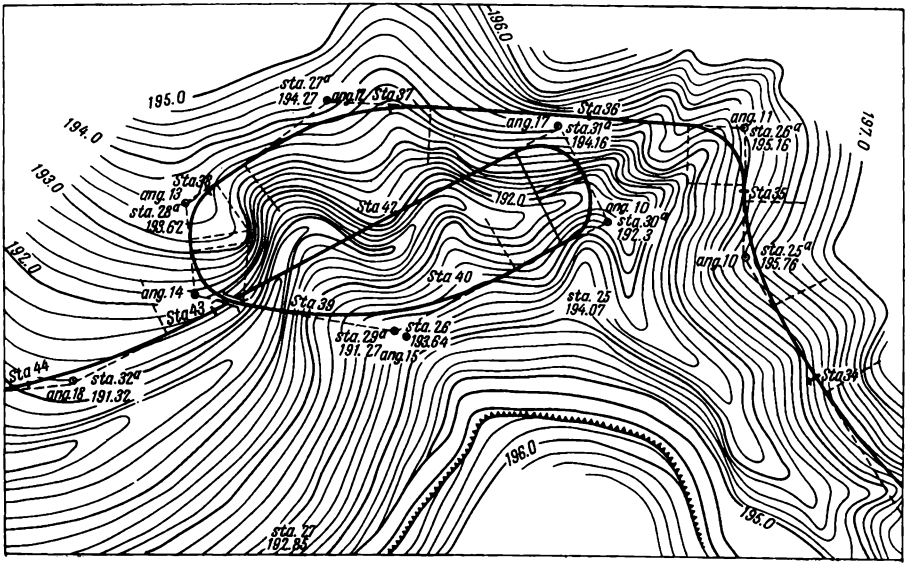


Fig. 194. Spiral route development (project by S. A. Treskinsky)

With a hillside slope of over 2 per cent, the practice generally followed up to recent times was to construct a cut-and-fill roadbed (Fig. 195). To ensure adequate stability of the embankment benches are made on the surface of the hillside with a height of 0.5 m and a length varying from 1.5 to 3 m, depending on the slope. These benches are given a gentle fall towards the hillside.

The side slopes of an embankment filled with stone not liable to weathering are designed with an incline of 1 : 1 to 1 : 1.3 with an embankment height up to 6 metres, and of 1 : 1 to 1 : 5 with

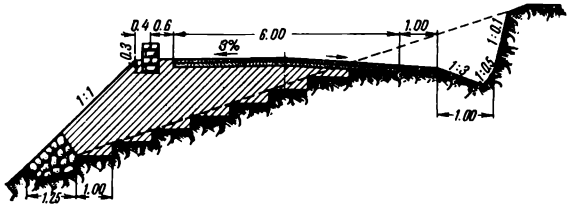


Fig. 195. Cross-section of road located on hillside

a height up to 12 metres. Whenever the fill material is coarse or medium-grained sand, gravel or crushed stone a slope of 1 : 1.5 is used.

The side slopes of cuttings in slightly-weathered nonfissured rock whose strata are not inclined toward the cutting are taken equal to 1 : 0.2, in granular-fragmental, gravelly and similar rock with large fragments, depending on the rock properties, nature of stratification and depth of the cutting—from 1 : 1 to 1 : 5; in other rock—1 : 0.2 to 1 : 1.5.

These recommendations concerning the slopes of a cutting are general for rock and uncemented materials other than rock, without subdivision according to types of soil. When determining the slopes of a cutting the following factors should be considered:

(1) in purely crystalline volcanic rock (granite, diorite, basalt, etc.), the frequency and direction of the fissures, the related forms of jointing and the degree of weathering of the rock;

(2) for sedimentary and metamorphic rock (limestone, sandstone, quartzite, crystalline schists, etc.), the mineralogical composition, the thicknesses of the separate layers, their geophysical constitution and mode of occurrence, i.e., horizontal, inclined or folded (Fig. 196).

In volcanic rock steeper slopes can be chosen when the rock is massive or has a comparatively dense network of horizontal joints

and a rare network of vertical ones, also when there is columnar jointing such as occurs with basalts.

In sedimentary rock the permissible side slope depends mainly on the direction and angle of incidence of the strata. If the strata

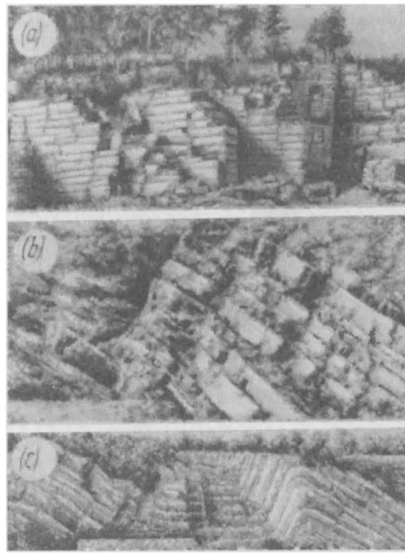


Fig. 196. Sedimentary rock structures:
a—horizontal; *b*—inclined; *c*—folded

are inclined towards the road, the slopes of a cutting should have a lesser grade and correspond to the inclination of the beds. If the strata are inclined away from the road, or are horizontal, the slopes may be near to vertical. When the kind and structure of the rock vary along the depth of the cutting the maximum grade of its slopes must also vary. Therefore, the data of the geological investigations carried out during the survey, and observations of the steepness and the state of existing slopes in the same conditions are of paramount importance for establishing the proper slope grades.

Ditches in hard rock usually have triangular cross-sections, the grade of the inner ditch slope being 1 : 3. In ground subject to disintegration, between the slope of the cutting and the outer edge of the ditch a berm is provided having a minimum width of 0.5 m, and the ditch is given a trapezoidal section with slopes of 1 : 1 to 1 : 0.5. The depth and the width of ditches at the bed level are selected according to hydraulic designs. In nonuniform soils the grade of cutting slopes is varied to correspond to the stability of the soil strata.

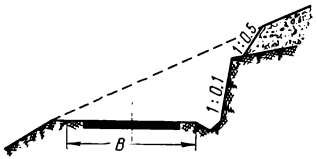


Fig. 197. Cross-section of a bench type cutting

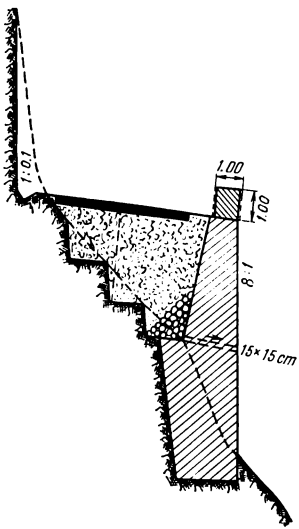


Fig. 198. Cross-section of road with a retaining wall

The design of a cut-and-fill roadbed on a hillside involves comparatively small earthworks. However, when constructing a roadbed of this kind on a steep mountain slope, a great amount of the soil is lost, as it slides uselessly down the slope. The design of benches on a slope often does not give sufficiently reliable adhesion between the fill and the natural slope, and in consequence the embankment gradually shifts. The settlement of the fill portion of the roadbed sometimes causes the appearance of longitudinal fissures in the pavement. That is why it is recommended at present to locate roads on stable hillsides with a slope exceeding 1 : 3 entirely in a cutting, on a sort of a bench. A cross-section of the bench type (Fig. 197), although entailing some increase in the earthworks, ensures the complete stability of the roadbed, if, of course, the hillside itself is stable. The general relation between the width of a cutting in the hillside and the width of the roadbed is as follows:

Average hillside slope, deg	15	20	25	30	35	over 40
Width of cutting into the hillside in relation to the total width of the roadbed, per cent	45-60	50-65	60-70	70-80	80-100	100

The carriageways of class I-III roads should be situated within the limits of the bench cut in the hillside.

On steep slopes, over 30-35 degrees, the earthworks involved in constructing the embankment increase substantially, because the

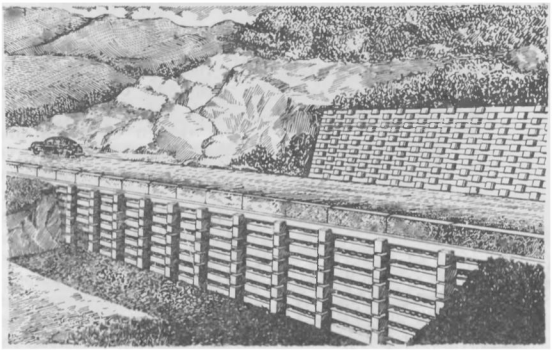


Fig. 199. Retaining and enclosure walls

slope of the latter is located at an acute angle to the natural slope. In this case *retaining walls* are necessary to support the embankment.

To decide whether or not the roadbed should be constructed with a retaining wall, the costs should be carefully compared. It should be remembered that an embankment with a retaining wall has a greater stability.

Frequently retaining walls are also built on a less steep hillside to increase the stability of the roadbed. A cross-section of a roadbed with a retaining wall is pictured in Fig. 198. Retaining walls are built of stone, concrete and reinforced concrete.

With retaining walls up to 4 m high and built of large slab-shaped stones the walls may be made of dry-built masonry, but if the height is greater the rule should be to use mortar.

The slopes of cuttings in marl and shale rock are easily weathered, and eventually disintegrate and crumble. To protect the slopes from atmospheric action they are faced with *protective* or *enclosure*

walls. The enclosure walls only cover the slope and do not support it as retaining walls do and, consequently, have smaller dimensions. Up to a height of 6 m enclosure walls are made of dry-built masonry. In seismic and landslide regions, however, and also when the roads are laid along the banks of watercourses, it is good policy to design crib walls of large reinforced concrete blocks (Fig. 199).

When inscribing cuttings into steep hillsides in stable rock faces, the rock may be permitted to overhang the road. Such a cross-section is called a *semi-tunnel* (Fig. 200).

If the road is located across a steep mountain slope, the retaining walls may have to be of a substantial height; in such cases, in order to reduce the quantities of work *semi-bridges* may be built, when part of the roadbed is situated on masonry or concrete vaults.

Fig. 200. Semi tunnel

On precipitous slopes, where shifting of the route into the hill will lead to enormous rockworks which would substantially increase the cost of road construction, and where a semi-tunnel cannot be built owing to the geological structure, *platforms* are usually cantilevered out of the rock, on which the roadbed is partially located (Fig. 201).

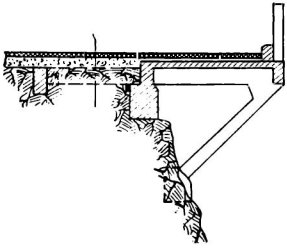


Fig. 201. Reinforced concrete plat form

To collect the water flowing down the slopes, *hillside intercepting ditches* are arranged at a minimum distance of 5 m from the edge of the cutting.

The lateral grades of the carriageway and the shoulders are determined according to the type of pavement selected. To ensure traffic safety on horizontal curves whose centre is located outside of the hillside, the carriageway is frequently given a crossfall of 1 per cent

towards the hillside, even though the super-elevation should have been designed with an outward crossfall directed down the slope.

On all steep hillsides, in the interests of traffic safety, *wooden, stone and concrete guard posts* or *parapets* are erected (Fig. 202). The inner sides of the parapets are painted white in order to improve

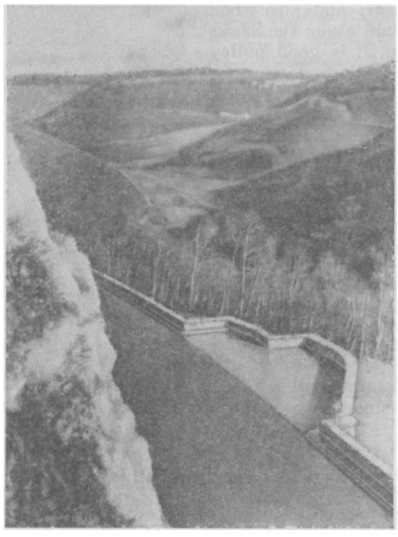


Fig. 202. Parapet and turnout on a mountain road

their visibility at night. *Metal posts* between which flexible metal strips are installed can be used as guards.

The embankment slopes, and sometimes those of the cuttings, are stabilized with stone. Such stabilization should be given special attention in river valleys where there is a danger of undermining and washout. In this case the lower part of the embankment is stabilized with a filling of coarse stones, gabions or by building retaining walls.

121. Mountain Road Profile

One of the main characteristics of mountain road profile design is the location of the roadbed along a hillside. Depending on the hillside slope, one and the same elevation difference along the road centre line may require extensive cuttings or embankments, and even the construction of retaining walls. Therefore a profile showing only the elevations along the roadbed centre line cannot in itself fully indicate the design of the road and the earthworks involved in its construction.

When designing a profile it is necessary at the same time to draw cross-sections of the land and to project the roadbed onto them according to the profile elevations. In this way the designer tries to obtain a profile which will permit the route to be located with the permissible gradients. At the same time the cross-section should ensure the minimum earthworks or the most stable arrangement of the roadbed, without the construction of expensive retaining walls.

The profile elevations are rectified by two methods: by horizontal shifting of the route according to the results of cross-section analysis, and by altering the magnitude of the gradients in the profile.

The first method involves an alteration of the number and situation of turning angles. In constricted circumstances shifting of the route will involve adjustment of the adjacent sections. Cases may arise when improvement of a given section may detract from the design of adjacent sections.

Alteration of the grade line according to the second method, without altering the horizontal route location, is also a very complicated task, especially with constricted conditions in the profile, since a substantial part of the adjacent sections will have to be designed anew. Also, lowering of the grade line in order to reduce the volume of embankments may be the cause of an increase in the volume of cuttings on neighbouring route sections.

Consequently, profiles and cross-sections for mountain roads should be designed during the progress of the survey, as this permits any requisite modifications of alignment to be accurately surveyed. If the design is not worked out during the field work, then the route location is altered on the basis of a contour map. Should this prove inadequate, several alternatives of the grade line are drawn up.

The grade line is plotted on a cross-section by means of transparent celluloid roadbed patterns. The cross-sections are usually drawn to a scale of 1 : 100 or 1 : 200. The retaining walls can also be plotted on the cross-sections by means of patterns. The areas on the cross-sections are determined by means of a planimeter, by division into geometric figures, and by other means. When plotting the cross-

sections and establishing the grades of slopes account should be taken of the local geological structure of the terrain.

When designing the profile, it is necessary to reduce the maximum longitudinal gradients when they coincide with tight horizontal curves. According to the road designing standards followed in the U.S.S.R. it is recommended to reduce the longitudinal gradients on tight curves as follows:

Radii of curves, m	50	45	40	35	30	25	20	15
Deduct from maximum longitudinal gradient, per cent	1	1.5	2	2.5	3	3.5	4	5

The reduction of the longitudinal gradients is essential for the following two reasons:

1. When travelling along a curve a sideway force is induced by the action of radial acceleration (see Sec. 20). The greater the lateral force, the larger becomes the yaw or slip angle between track and wheel. The wheel slip causes additional resistance to movement and increases fuel consumption and tyre wear. It has been experimentally established that the centrifugal force increases the power necessary for negotiating a curve. As is known, the power, required for overcoming rolling resistance is

$$N\frac{fGV}{270}\ \text{hp} \tag{237}$$

where f = factor of rolling resistance
G = vehicle weight, kg
V = vehicle speed, km/hr.

Assuming a constant speed and vehicle weight, the power required increases owing to the increase of the factor of rolling resistance. The maximum yaw or slip angle occurs at the maximum value of the sideway force coefficient equal to 0.15-0.20. The slip angle for passenger vehicles with a sideway force coefficient of 0.15 is shown on the diagram (Fig. 203*a*). The maximum slip angle is observed for the ZIL-110 car, amounting to 2.8 degrees, and it is slightly less for the GAZ-12 cars, being 2.2 degrees. Assuming the average value of the slip angle for passenger cars equal to 2.25 degrees, then according to the diagram in Fig. 203*b* it can be seen that with this slip angle the factor of rolling resistance increases by about 50 per cent. Thus, in order to provide on tight curves the same conditions that obtain on straights, it is necessary to reduce the longitudinal gradient by a value equal to half the factor of rolling resistance.

2. On tight curves vehicles moving up the slope on the inner side of the roadway have to overcome an additional gradient because of the shorter length of the curve. The magnitude of the additional

gradient is

$$\Delta i = i_c \left(\frac{R_c}{R_t} - 1 \right) \tag{238}$$

where i_c = longitudinal gradient along the road centre line
R_t = radius of the vehicle trajectory, m
R_c = radius of curve centre line, m.

If it is assumed that when moving along the curve, the centre of the vehicle trajectory is located at a distance of 1.5 m from the

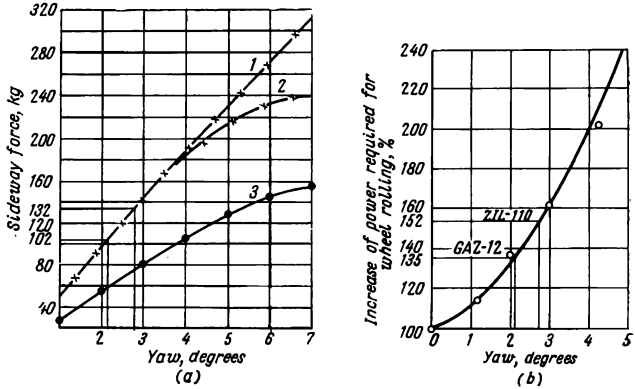


Fig. 203. Wheel yaw:
a—relation to sideway force; b—increase of power required for rolling the wheel; 1—ZIL-110 (7.50-16); 2—GAZ-12 (7.00-15); 3—Moskvich (5.00-16)

inside edge of the roadway, taking into consideration its widening, then the radius of the trajectory can be determined from the formula

$$R_t = R_c - \left(\frac{B_n}{2} + e \right) + 1.5 \tag{239}$$

where B_n = normal roadway width
e = additional width of the inside roadway lane.

It has been established on the basis of the above calculations that an additional longitudinal gradient of 0.1 per cent and over has to be overcome only with radii of 125 m or less, and its magnitude depends on the longitudinal gradient along the centre line (Table 39).

The increase of the longitudinal gradient is significant only for curves where the upgrade traffic follows the inner side of the roadway.

TABLE 39

Maximum longitudinal gradient, %	Curve radii, m							
	125	80	60	50	40	30	20	15
	Increase of longitudinal gradient, %							
9.0	0.2	0.3	0.4	0.5	0.8	1.2	2.3	2.7
7.0	0.1	0.2	0.3	0.4	0.6	0.9	1.8	2.1
6.0	0.1	0.2	0.3	0.4	0.5	0.8	1.5	1.8

In Figure 204 such conditions occur on curves Nos. 1 and 3; on curve No. 2 the movement along the inner side of the curve is downwards.

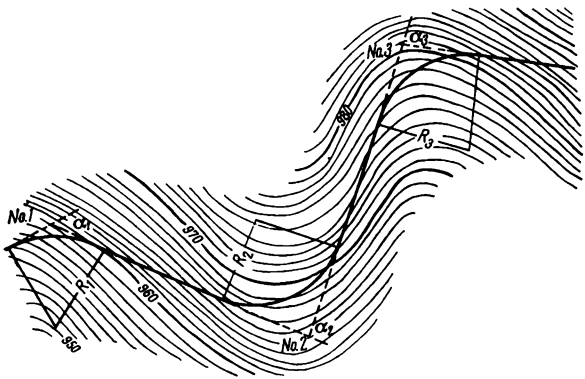


Fig. 204. Traffic conditions on curves:
a—on curves Nos. 1 and 3 the up-grade traffic moves along the inner curve of the roadway; b—on curve No. 2 the up-grade traffic moves along the outer curve

Thus, for curves on which the vehicles move upgrade along the inner lane of the road, the total resistance is composed of the additional factor of rolling friction, and the additional gradient due to the shortening of the vehicle's trajectory. The magnitude of the necessary reduction in the longitudinal gradient depends on the gradient and the type of surfacing (Table 40).

For curves with a radius exceeding 100 m in any conditions of traffic, and for those having a radius less than 100 m but with down-

TABLE 40

Type of surfacing	Maximum gradient, %	Curve radii, m							
		100	80	60	50	40	30	20	15
		Reduction of longitudinal gradient, %							
Intermediate	7.0	2.1	2.2	2.3	2.4	2.6	2.9	3.8	4.1
Intermediate	9.0	2.2	2.3	2.4	2.5	2.8	3.2	4.3	4.7
Inferior	9.0	2.6	2.6	2.7	2.8	3.1	3.5	4.6	5.0

grade traffic on the inner side of the curve, the required reduction in the maximum gradient is given in Table 41.

TABLE 41

Road class	Curve radii, m	Magnitude of required reduction in maximum gradient related to type of surfacing, %			
		Heavy-duty	High-quality	Intermediate	Inferior
II	600-400	0.5	—	—	—
III	400-250	—	1.0	—	—
IV	250-125	—	1.0	2.0	—
V	125-60	—	—	2.0	2.3
IV-V	60-15	—	—	2.5	3.0

The longitudinal gradient should also be eased off on the approaches to the curve at a distance of 5 to 10 m on each side. Designers should avoid using long sections having maximum gradients above 6 per cent. Where such gradients occur inserts having maximum gradients of 2 per cent and a minimum length of 60 m should be used. Not more than one insert should be introduced per kilometre of continuous steep gradient.

To ensure traffic safety tight curves should never be used at the lower end of long downgrades.

In exceptional cases, with altitudes of the terrain less than 3,000 m, the maximum longitudinal gradient may be increased by 1.5 to 2.0 per cent on short stretches up to 0.5 km long, in order to reduce the earthworks and the cost of construction. The increase of the maximum gradient should be justified by engineering and economic calculations.

Simultaneously with the design of the profile all the drainage problems must be solved. Apart from structures, additional items such as intercepting, side and diverting ditches are designed. The drainage layout is indicated on the land contour map, and the ditch cross-sections determined by hydraulic calculations. With a view to the considerable slopes and the appreciable velocities of water flow, it may prove necessary to provide for paving of the ditch beds and slopes (with the exception of rocky ground). It is good to render the paving with cement mortar in order to prevent the percolation of water into the subgrade, which is especially dangerous in intercepting ditches. For this reason the intercepting ditches should never be designed with considerable gradients. The water from these ditches should be discharged away from the road and into thalwegs or depressions, but not into road ditches.

122. Route Location over Talus

Under the action of weathering, rock will gradually disintegrate, weathering being most intensive on slopes facing South. As a result, detached fragments, both large and small, roll down and accumulate at the foot of mountain slopes to form *talus*. Talus occurs most frequently with fissured and stratified soil.

On gentle slopes talus is deposited at the angle of natural repose, from 28 to 35 degrees, depending on the coarseness of the particles. The coarser particles are formed upon the disintegration of volcanic rocks, while the fine ones originate from shale. Talus usually has a characteristic fan shape and its tail (the lower portion) may cover an extensive area around the foot of the slope (Fig. 205).

In the U.S.S.R. talus is classified according to the potential mobility of the slope, which depends on the structure of the talus, the mobility ratio $\frac{\alpha}{\varphi}$ (where α is the dip slope of the talus surface and φ is the angle of natural repose for the materials composing the talus) and on the rate of supply of the weathered product. When the mobility ratio is equal to unity, i.e., when $\alpha \quad \varphi$, the talus is classified as mobile and, therefore, unstable. In this case, measures ensuring the stability of the roadbed must be provided for.

If the angle α is less than φ and the mobility ratio varies from 0.7 to 1.0, the talus is classified as fairly mobile, and it is also necessary to ensure roadbed stability by means of special structures.

If the mobility ratio is within the range of 0.5-0.7, the talus is slightly mobile, and with the mobility ratio below 0.5 the talus may be considered as relatively immobile and it can be used for locating the roadbed without additional structures, except for talus incorporating silty clayey materials.

When the strip between the talus and the watercourse is sufficiently wide the route can be located below the talus with a retaining wall to protect the road. These walls must be built up as the talus accumulates. The annual growth of the latter depends on the steepness of the slope, the strength of the rock and on the climatic conditions.

Fig. 205. Location of route on lower part of talus

Substantial difficulties for road design occur when an extensive talus overflows into a watercourse. In this case the route must be transferred to the opposite bank of the valley, and this will require the building of two bridges. The final route is selected by technical and economic comparison of the alternatives.

If the talus is composed of coarse permeable rubble the route can be located across it. If the talus has become stabilized and a mantle of soil has appeared on its surface that is covered with vegetation, the road is designed using normal techniques. When the talus is active, owing to the accumulation of deposits from the upper part the talus train creeps down to the watercourse. In this case, two retaining walls have to be designed: one for protecting the road from a rockfall from above, and the other below, for stabilizing the roadbed against shifting together with the body of the talus (Fig. 206). The retaining walls should be taken down to bedrock. With an extensive talus depth the dimensions of a retaining wall

may be very considerable. In all cases it is necessary to investigate the possibility of erecting a retaining wall at the head of the talus, where the products of soil destruction originate and where it is the narrowest. In certain cases ground water may emerge in the talus, as well as surface water runoff. Under the influence of flowing water, and sometimes as a result of the land relief, the whole talus may move down the slope.

Cases are known when a road laid on such a talus has crept several metres down the slope during a year. It is very difficult to stabilize a mobile talus chute. Therefore, when crossing a narrow mobile talus it is good to build a bridge. When a wide mobile talus occurs, the possibility of transferring the route to the other bank of the valley should be investigated, or even the possibility of building a tunnel.

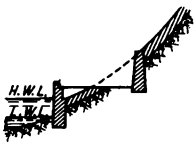


Fig. 206. Location of road over talus chute

If the talus consists of granite, porphyry or limestone, then, in certain cases, these materials can be used advantageously in filling embankments, constructing the roadbed or manufacturing concrete.

In a number of cases, instead of building upstream retaining walls, it will be better to remove the talus and use the material for the construction of embankments between the talus fans.

123. Route Location over Silt Washout Fans

The accumulation of great masses of rock debris on the steep slopes of canyons and ravines may call forth the appearance of *mud* and *stone flow streams* after periods of heavy and prolonged rainfall. These streams comprise a mixture of water-saturated soil and stones with a unit weight of 1.2 to 1.6 ton/m^3. The total quantity of mud and stone materials may reach enormous amounts.

The formation of mudflows is facilitated by the accumulation of extensive masses of rock debris on the slopes of canyons and ravines, and also by steep gradients on hillsides and thalwegs. The water running down such slopes with a high velocity erodes and washes out the thalweg banks.

The formation of mudflows is also caused by the imprudent destruction of forests and scrub on the slopes, which hitherto retained the soil mantle with their roots, resisted weathering and erosion, and retarded the flow of surface water.

In mudflows the stones are partly carried along in a suspended state while the large ones roll down the thalweg bed. Separate stones are caught up by the irregularities of the underlying rock and dam

the flow. However, the inflow of debris from above breaks the barrier and the mud stream then flows downward with a still greater velocity. It has been established that the linear dimensions of the particles carried along by the flow are proportional to the square of the velocity, while the weights of the particles are proportional to the sixth power of the velocity. For this reason in mountain rivers, and more so in mudflow streams, stones of appreciable dimensions are carried down.

The velocity of mudflow streams can be expressed as

$$V = mH_{av}^{2/3} i^{1/2} \text{ m/sec} \tag{240}$$

where H_{av} = average depth of the flow, m
i = bed gradient
m = factor depending on the average diameter D of the mudflow deposits.

Values of D	10	20	50	100	200	500
Values of m	19	17	15	13	12	10

To determine approximately the velocity of a mudflow stream in relation to the stone material size, the following formula can be used

$$V = 5.30\sqrt{D(1-0.01p)} \text{ m/sec} \tag{241}$$

where D = average size of the particles in the flow, m
p = content of solid materials in the mudflow stream, expressed as a percentage by weight.

The velocity of a mudflow stream can be approximately computed by the formula $v = s\sqrt{D}$ m/sec.

Having established the mudflow stream water level and its velocity, the mudflow discharge can be determined.

In practice designers estimate mudflow discharge by means of the empirical formula

$$Q_m = (k_1 m_1 m_\mu k_2 + 1)\, Q_w \text{ m}^3\text{/sec} \tag{242}$$

where k_1 = factor taking into account the increase in runoff during mudflow and equal to 1.1
m_1 = factor depending on the longitudinal gradient i and determined according to the formula

$$m_1 = 3 - \frac{0.3}{0.1 + 4.8 i^{1.4}}$$

m_μ = factor depending on the area of mudflow formation in the catchment basin, which is determined from the formula

$$m_\mu = 0.425 - \frac{0.04}{0.01 + 10\mu^2}$$

the coefficient μ being the ratio of the mudflow-forming area to the whole catchment area

Q_w = maximum water discharge, m^3/sec

k_2 = factor taking into account the ratio between the volume of debris which remains on the debris fan and the volume of floating debris deposited beyond the debris fan. For mudflow streams $k_2 = 1$.

The volume of the solid discharge, i.e., the quantity of solid material discharged by the mudflow stream during the flood period, is determined according to the formula

$$W_s = k_1 m_1 m_\mu W_w \qquad (243)$$

where W_w volume of water discharged during the flood period.

If the mudflow stream cannot be bypassed, then it is better to cross it at the narrowest point having stable rocky banks.

The bridge is designed with one span, and protective structures must be provided upstream and downstream of it. Simultaneously measures are taken to reduce and eliminate mudflow. To decrease the intensity of the erosion process, the uncontrolled felling of trees and the destruction of scrub on the slopes must be forbidden and correct cultivation of the soil introduced. For the same purpose, trees and shrubs are planted, the slopes are terraced, and drainage and discharge ditches constructed. To reduce the energy of a mudflow stream and retain the debris, a system of special dykes (barrages) is constructed across the stream bed. The greatest success will be achieved by the combined use of all these arrangements.

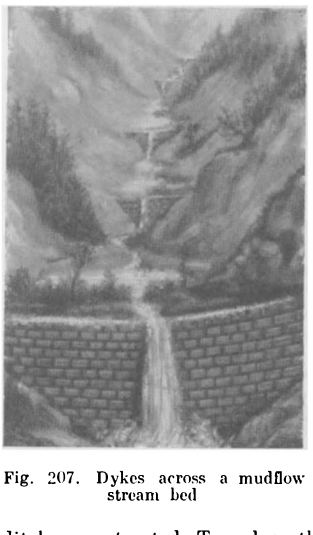


Fig. 207. Dykes across a mudflow stream bed

A transversal dyke is a stone or concrete wall, 2 to 5 m high and with a special profile. The dykes are so placed along the thalweg that the gradient of the line connecting the foot of the upstream dyke and the top of the downstream one is at most 6-8% (Fig. 207).

On the upstream side and, especially, on the downstream one strong fortification against washout must be provided.

The upper surface of the dyke is made concave, with an average slope toward the centre of 1.0 to 2.0 per cent.

When the mudflow streams are crossed by roads of inferior classes the building of a bridge with large openings is not advantageous. With a small traffic intensity it is possible to direct the mudflow stream along a special apron which is made on a level with the

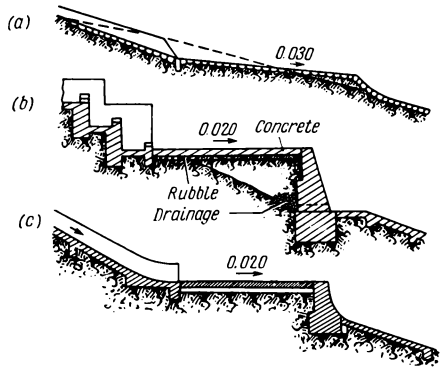


Fig. 208. Designs of aprons for the passage of mudflow streams over a road:
a—reinforced with cobble-stone paving; *b*—concrete apron with approach bed; *c*—concrete apron with chute

carriageway (Fig. 208). On roads with a heavy traffic intensity, where there may be places at which comparatively small mudflow streams are to be crossed, but which approach the road at an appreciable gradient, mudflow bridges are designed which carry the mudflow over the road.

The bridge is built of stone. The profile sections and a general view of such a bridge built on a highway in the U.S.S.R. are shown in Fig. 209. On the approaches to the bridge a chute was built designed to accommodate a flow which occurs statistically once in 100 years and amounts to 18 m^3/sec. The artificial bed is made of cemented hard stone. In the upper part of the mudflow bridge the bed gradient is 1.3 per cent, and at the end of it 4.38 per cent. The mudflow stream is discharged from the bridge into the river.

In certain cases the road, although located at an appreciable distance from mountain slopes, is still within the zone of mudflow debris deposits. Consequently, it will always be in danger of being

blocked or destroyed, since debris may be deposited at a distance of several kilometres from the mountain slopes. In these cases it is expedient to design continuous or intermittent debris retaining dykes on the upstream side. Continuous dykes are recommended when the

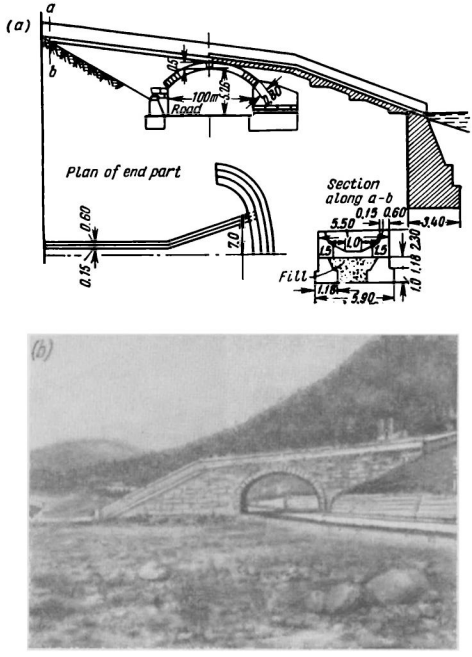


Fig. 209. Mudflow bridge:
a—sections; *b*—general view

width of the bed exceeds 100 m. They are aligned at right angles to the stream flow. The debris is deposited at the dyke, but the water flows around it and through a structure on the road. The length of the dyke depends on the width of the stream bed and on the coarseness of the deposited particles and can be determined from

the formula

$$l = \mu B \tag{244}$$

where μ = factor allowing for bed contraction by the dyke
B = width of bed, m.

The contraction factor is determined in accordance with the accepted percentage m of debris retained by the dyke and with the relative coarseness of the debris, $\frac{d_{av}}{B}$, given in Table 42.

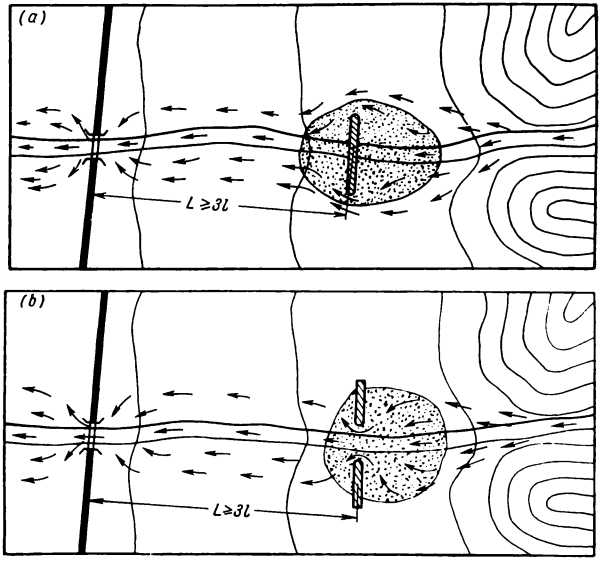


Fig. 210. Debris retaining dykes:
a—continuous; b—intermittent

The distance between the dyke and the road is selected in accordance with the quantity of debris and with the distance of the road from the mountain slopes. If the dyke is situated next to the mouth of the mudflow stream, it may be destroyed by large boulders. However, its distance from the road should be sufficient to permit the debris to be deposited before reaching the road structure.

TABLE 42

Percentage of retained debris m	Contraction factor at $\frac{d_{av}}{B}$	
	<0.01	>0.01
100	0.55	0.65
75	0.40	0.45
50	0.30	0.35

It is good to locate the dyke at a distance from the road of at least three times the length of the dyke.

Intermittent dykes are used when the stream bed is not wider than 100 m. Their total length is determined according to the above formula, and the size of the openings is calculated for the discharge of a water flow having a given frequency of reoccurrence.

The dyke cross-section is made trapezoidal with a width at the top of 0.5 to 2.0 m, depending on the material and the strength of the mudflow stream. Figure 210 pictures alternative arrangements of debris retaining dykes.

124. Measures for Control of Landslides and Falls

When designing highways in mountainous and broken country it is necessary to give regard to the possibility of violation of the natural hillside and road slope stability under the action of unfavourable natural conditions. In some cases the stability may be disturbed if during the design of the road the topographical, geological and hydrogeological conditions have not been fully taken into account. In such cases the construction of the road may disturb natural slope stability, while the movement of traffic over the completed road may lead to slope deformation. Because of these circumstances it is imperative that, when designing a road on a hillside known to exhibit unfavourable natural conditions, measures be taken to improve the stability both of the whole slope and of the roadbed.

The forms of hillside and slope stability failure are very diverse. Prof. N. N. Maslov proposed the following classification of these forms:

(1) *falls*, observed on steep precipitous ledges in rock having a highly developed system of jointing (Fig. 211*a*);

(2) *slump with shear and rotation*, which occurs mainly in generally uniform rock with laminations and with excessive slope grades (Fig. 211*b*);

(3) *shear with subsidence*, which may take place if in the rock mass there occurs a stratum of soft clay, silty sand, loessial soil, etc. (Fig. 211*c*);

(4) *sliding* (*glissades*), shear occurring along bedding planes, faults, etc.; observed with a clearly visible slip surface sloping down the hillside (Fig. 211*d*);

(5) *landslide*, when the shift occurs as an almost horizontal displacement along a soft plastic clayey bed with a small grade, owing to lateral pressure (Fig. 211*e*);

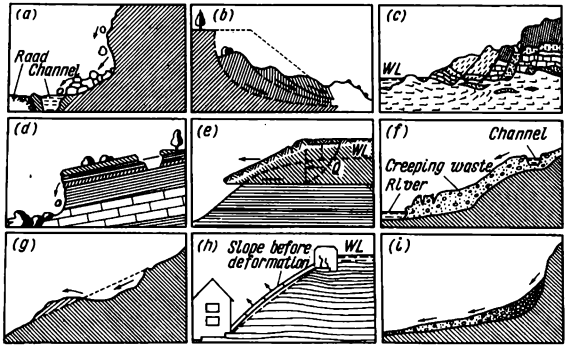


Fig. 211. Forms of slope flow and failure (according to N. N. Maslov)

(6) *creep*, sliding over a surface of basement rock (Fig. 211*f*);

(7) *flow*, a surface slip of soil masses having an excessive moisture content (Fig. 211*g*);

(8) *plastic deformation* of a slope observed in argillaceous rock and characterized by the low rate of creep—centimetres per year (Fig. 211*h*);

(9) *secular reworking* of a slope owing to atmospheric action (heating and cooling, freezing and thawing, wetting and drying); talus is a typical example of such slope reworking (Fig. 211*i*).

However, it must be recognized that in this classification all the forms of slope stability violation are given in the *pure sense*, while in nature several of the forms usually occur simultaneously. This substantially complicates the nature of action to be taken to ensure slope stability. The greatest difficulties are encountered when roads have to be located through regions prone to *landslides*.

Landslides take place owing to violation of the equilibrium of the earth masses and occur without overturning of the moving rock. The rapid displacement of earth masses, accompanied by overturning and breaking up of the rock, is called a *fall*.

Falls usually occur on steep rocky hillsides consisting of weathered rock with inclusions of separate large stones. Overhanging rock hillsides often present a fall hazard, and the road should not be located in such places. To avoid rock falls, during road construction the potentially unstable large stones on slopes must be removed, the slopes levelled and a network of drain ditches arranged to reduce hillside erosion. When the falls occur repeatedly special protective galleries must be constructed of the type used for protection against avalanches (see Sec. 125).

Landslides are extremely dangerous, since they destroy roads, structures, buildings and occasionally entire villages. For the successful control of landslides, the causes which originate them should be studied and the correct action taken for stabilizing the slopes.

Surface and ground water are the main causes of landslides. Surface water, which accumulates in depressions having no outfall, penetrates into and saturates the soil layers. As a result, plastic landslides or creeps may occur on the slopes. In fissured rock the surface water frequently runs down the fissures to the impermeable soil, reducing soil cohesion. Landslides tend to develop where inclined stratification, fissures or clay bands are present. In areas prone to landslides parallel ravines on the surface, soil strippings and faults, and slide hummocks and terraces at the foot of the slope may be observed. Trees on the slide slopes grow at an angle.

Landslides may originate when a roadbed is erected on unstable slopes, when the excavation of a cutting may cause a shift of the higher inclined rock strata, while the construction of an embankment may lead to shear of the underlying soil mass. Landslides may also be caused by washout of the foot of a slope on river banks and sea shores.

In regions where landslides have occurred or are possible, the route should be located above landslides whose causes are on the downstream side, or below them when the causes are on the upstream side. With a combined type of landslide it is better to locate the route on the upstream side. The route layout is selected after detailed investigations of the slopes in the area, during which the landslide morphological features, the hillside geological texture, the steepness or dip of rock bedding, and the composition and types of rock and depositions are studied; also, the causes of landslide formation are established, and the water bearing strata, ground water inflow and direction of runoff are determined. The movement and the development of a landslide should be investigated by observing the position in plan

and in height of special interrelated survey stations and bench marks installed both within the body of the landslide and beyond its limits.

On the basis of the data collected the causes of origin are established, preventive action is selected to eliminate these causes, and the route location is chosen.

When the road is designed on slopes prone to landslides, it is good practice to design the roadbed as a cut-and-fill cross-section and to avoid pure cuttings. It is not recommended to completely cut through the talus deposits at stream mouths. It is not good to design high embankments, since the additional load on the slope may cause a landslide.

When working out anti-landslide measures every effort should be made to eliminate the main causes which originate landslides. The control of only the consequences of landslides will give only short-time results, inasmuch as it does not exclude the recurrence of landslide development in the future.

Among the anti-landslide measures of a *preventive* character are the preservation of trees and shrubs, adherence to agricultural rules, and correct methods of drainage. Also, works which disturb the slope stability should be forbidden. However, all these measures will have a positive effect only if applied simultaneously with *active control measures*, which include:

(1) removal of surface water by means of intercepting and diversion ditches;

(2) removal of ground water by means of various types of drainage;

(3) drainage of the landslide body by means of ditches and drains;

(4) improving the adhesion of the sliding mass to the landslide bed by means of keys, poles and piles;

(5) erection of retaining structures such as retaining walls, abutments, banks, etc.;

(6) protection of the slopes against erosion and washout by the construction of dykes and coverings;

(7) stabilization of the soil with various binders, also by electrochemical methods and congelation.

The above measures are rational only when they are applied in combination.

Having established according to the data of the topographic survey a plan of the landslide area, and, according to geological cross-sections, the direction and depth of ground water flow, first of all action is taken to *divert all* surface and ground water from the landslide area. For this reason a comprehensive system of ditches and drains is created.

Intercepting ditches for collecting surface water are situated along the perimeter beyond the boundaries of the landslide area on the uphill side. The water is diverted into thalwegs and ravines situated

beyond the limits of the landslide area. It is recommended to design the intercepting ditches with a maximum gradient of 2 to 3 per cent, to avoid high water velocities and possible washout. The ditch section is determined according to hydraulic calculations, and the kind of stabilization depends on the water velocity.

When ditches of a large section are to be cut or a high water velocity is expected, concrete fortification is used in the form of special *aprons*. It is known that the deeper the ditch, the greater the stability of the slope is violated. Furthermore, the concentration of a large amount of water in an intercepting ditch is not desirable, since if the fortification is damaged the water may percolate into the soil, which will immediately detract from the slope water conditions. Therefore, intercepting ditches are designed in two or three rows with independent water diversion to beyond the limits of the area.

The ditches excavated in the body of a landslide, which serve for the rapid disposal of water from its surface and for the reduction of its percolation, are usually arranged as a well developed network. When designing such ditches special attention should be directed to the strength of their fortification. In such cases the paving is generally placed on a layer of gravel or sand treated with organic binders which will not permit the passage of water in case of damage to the paving. The state of ditches in service must be carefully watched, since a disrupted drainage network may cause greater damage than its total absence.

Drains for the interception of ground water are sited along the boundaries of the landslide area, while for drying the landslide mass they are also located in the landslide body.

It is good to locate *lateral drains*, which are generally at right angles to the direction of ground-water flow, on the part of the hillside that is not affected by the landslide, since an insignificant movement of the landslide may disrupt water collection. When diverting ground water through the body of the landslide, *longitudinal drains* are laid that direct the water to the toe of the slope or into a culvert or other structure. Longitudinal drains, which are located along a landslide, are less sensitive to its motion. At junctions or changes in the direction of the drains manholes or inspection wells are installed for observing the behaviour of the drainage structures. Figure 212 depicts a plan of a landslide area showing the measures to be undertaken.

Retaining walls were considered until recently as the main remedy, which did not require the elimination of the causes of landslides. This erroneous view prevented an appreciation of other measures and in certain cases led to the destruction of the retaining walls. At present retaining walls are used only in conjunction with other anti-

landslide measures to prevent the slopes from being washed out by water and also to support the landslide mass.

When erecting a retaining wall it is necessary to provide for drainage beyond the wall in order to divert the water. For increasing the resistance to shear and for drainage of the slope use is made of *buttress pillars* and *buttress benches*, which are located on the lower part of the slope.

The *electrochemical method* of reinforcing landslide slopes can be used with clayey soil. It consists in applying a direct current between electrodes inserted in the soil; this creates an electrolytic process, by means of which the physical and chemical features of the soil are altered (decrease of moisture content, increase of density, etc.).

When an electric current flows through saturated ground the water moves from anode to cathode, while the clay particles suspended in the water flow from the cathode to the anode. This phenomenon is the basis of the method, which makes it possible to dry the ground by electric drainage. The electrodes are steel pipes 20 to 30 mm in diameter.

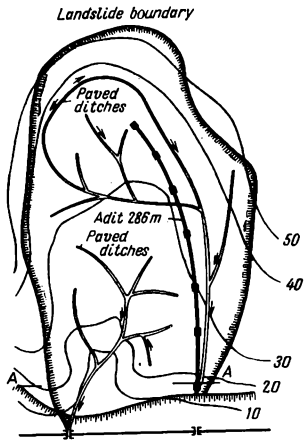


Fig. 212. Complex of anti-slide measures

Grouting or *cementation* of slide slopes is used in the case of fissured rocks, fissured clays and marls. The hardened cement mortar forms a skeleton in the body of the landslide mass and protects it against disintegration. The mortar introduced into the fissures fills them and prevents moisture from penetrating into the soil.

Silicification is used for reinforcing sandy soil by forming a silicic acid *gel* which is precipitated in the reaction of calcium chloride with liquid glass.

The main methods of landslide prevention used in road construction practice are the diversion of surface and ground water and the erection of retaining walls. The appreciable expense involved in the investigation and application of new and more modern methods for controlling landslides is amply justified, since the damage which may be caused by landslides is very great.

125. Protection of Road Against Avalanches

On highland roads located in regions with a great volume of snowfall *avalanches* are quite frequent. Avalanches are snow masses which have lost their adhesion to the underlying material and which, in consequence, move or fall down the slope. Avalanches are of an enormous size, reaching several tens or even hundreds of thousands of cubic metres, and fall with great velocity, destroying roads and road structures. During their fall avalanches create a strong air wave which precedes them and causes destruction in places which are not directly reached by them.

A danger of avalanches frequently appears on leeward slopes, where masses of snow accumulate in the shape of cornices hanging over the sides of the mountain.

There are various causes of snow avalanches. The snow covering may not be a homogeneous continuous mass, but stratified. The layers of snow of variable density are frequently divided by crusts of hard frozen snow. The snow density varies between 0.04-0.05 (fresh snow) and 0.7-0.8 (wet snow). Thus, the weight of the snow mass varies widely, from 40-50 to 700-800 kg/m^3. The snow density increases towards the end of the winter owing to a recrystallization process involving a constant growth of the snow crystals. Owing to the considerable difference (10 to 15 deg C) between the temperatures of the top and lower layers of the snow covering, a movement of water vapours is created from the lower layers to the top ones. As a result the crystals are destroyed and the lower layers become loose. In the top layers, on the contrary, the crystals consolidate and a frozen snow crust is formed. Thaws occurring in winter create a series of frozen crusts in the snow covering between which powdery snow is interleaved. Thus, all these features of snow accumulation lead to a gradual loss of adhesion between the layers and, therefore, to the formation of avalanches.

Avalanches are sometimes composed of dry snow which has a minimum adhesion. Falls of overhanging cornices formed of dry snow are particularly frequent. Such falls may be triggered off by a gust of wind, a shot or even by loud talking. Dry snow is intensely pulverized when it falls, forming a snow cloud which moves downward at a great velocity.

In spring or during extensive thaws, avalanches composed of wet snow may occur. The underlying layers of snow in such an avalanche become saturated with water, which reduces the adhesion to the earth's surface, and the snow mass slides down the slope. A wet avalanche moves as a solid mass, carrying along stones and trees which are broken by its movement.

When designing mountain roads one has to determine the regions which are dangerous from the point of view of avalanches. This can be done by studying cartographic material or aerial photography data, and also by direct survey on the site.

Avalanche regions are characterized by steep ravines and thalwegs which have a depression at the head, a gathering ground in which the

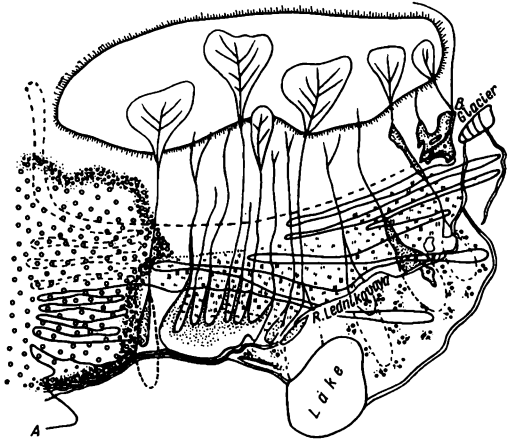


Fig. 213. Route alternatives in a region prone to avalanches (according to G. K. Tushinsky)

snow collects. The slopes facing South present a special danger, since at that side numerous frozen crusts with a slippery surface will develop in the snow, and in spring the snow thaws rapidly.

In the design stage care should be taken to prevent crossing places which present an avalanche danger, or to reduce the number of such crossings.

Figure 213 shows two route alternatives in a region prone to avalanches. The first alternative (the solid line) is a route using the whole mountain slope for its development, but crossing several times places which present a danger from avalanches. In the second alternative (the dotted line) the route is developed within the limits of a forest area which does not present a danger of avalanches, and which crosses the site prone to avalanches only once.

On very steep slopes (over 60 deg) great snow accumulations do not occur since the snow slides down gradually. Slopes having extensive snow accumulations which are in a state of unstable equilibrium are the most dangerous ones. The critical angle of inclination depends on the physical and mechanical characteristics of the snow covering, the average being 22 to 24 degrees.

The maximum depth of the snow cover which will remain in equilibrium can be determined as follows: Let us suppose (Fig. 214) that a snow layer with a depth h has failed along a slope inclined to the horizon at an angle α. For calculation purposes let us introduce the following notation:

γ = unit weight of snow, kg/m³
l = length of the plane of failure, m
c = force of cohesion along the slide plane, kg/m²
n = ultimate tensile strength, kg/m²
f = coefficient of friction.

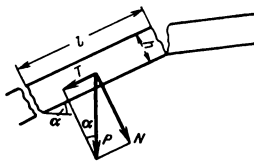


Fig. 214. Avalanche force diagram

The calculation is made for a strip one metre wide. In this case the weight of the broken-off section of snow is $P = \gamma hl$ and the shearing force $T = P \sin \alpha$. The retaining forces consist of the adhesion forces along the slide sections, the friction resistance and the ultimate strength of the upper part. In the state of limiting equilibrium the following condition should be satisfied:

$$P \sin \alpha = \gamma hl \sin \alpha + cl + fhl \cos \alpha + nh \tag{245}$$

Whence

$$h = \frac{\frac{c}{\gamma}}{\sin \alpha - f \cos \alpha - \frac{n}{\gamma l}} \tag{246}$$

Thus, it is possible to find the maximum depth of the stable snow covering for slopes of varying gradients. The above formula can also be used for determining the maximum gradient of the land on which avalanches will not occur with any depth of snow cover. The value of $\cos \alpha$ is determined from the formula

$$\cos \alpha = \frac{-fn \pm \sqrt{\gamma^2 l^2 (1 + f^2) - n^2}}{(1 + f^2) \gamma l} \tag{247}$$

The quantity n^2 is very small in comparison with the other members and can be neglected. Thus the formula becomes

$$\cos \alpha = \frac{\gamma l \sqrt{1 + f^2} - fn}{(1 + f^2) \gamma l} \tag{248}$$

The results of calculations have shown that the slide angle α varies from 23° (fresh snow) to 31° (wet snow). However, with the appearance of thaw water and due to other reasons avalanches may occur with angles of 15 to 18°. This has been frequently observed on mountains. To control avalanches snow accumulation is reduced in avalanche regions by erecting snow fences on the slopes of catchment areas; by forbidding the felling of trees and grubbing up of

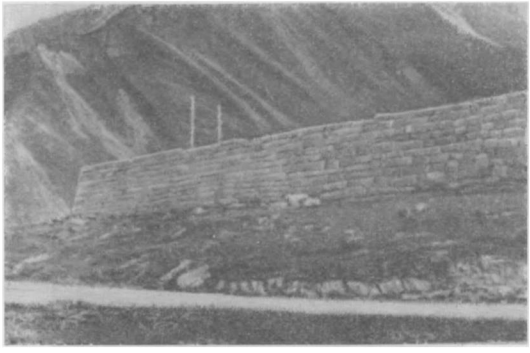


Fig. 215. Snow retaining wall

shrubs; by terracing the slopes, by constructing retaining walls and erecting fences to keep the snow from sliding on the slopes; by building diverting and protective structures such as retaining walls and avalanche breakers, which channel the sliding snow masses into side ravines and depressions on slopes; and by erecting snow retaining galleries.

Retaining walls which are designed to intercept avalanches are located at right angles to the direction of the avalanche (Fig. 215).

Diverting masonry structures are triangular walls of dry masonry 4 to 6 m wide and 5 to 10 m high. The length of these structures may attain 30 to 40 m.

Avalanche breakers are made of stone or concrete in the form of triangular dykes at an angle of 30 to 40° to the avalanche.

Buildings and bridges are protected with dykes which are substantial retaining walls or earth structures with reliable slope stabilization.

On particularly dangerous sections special protective structures, called *galleries*, are made above the road. In Fig. 216 a characteristic funnel can be seen in which snow accumulates, and along which an avalanches will occur. Galleries permit the snow mass to slide over

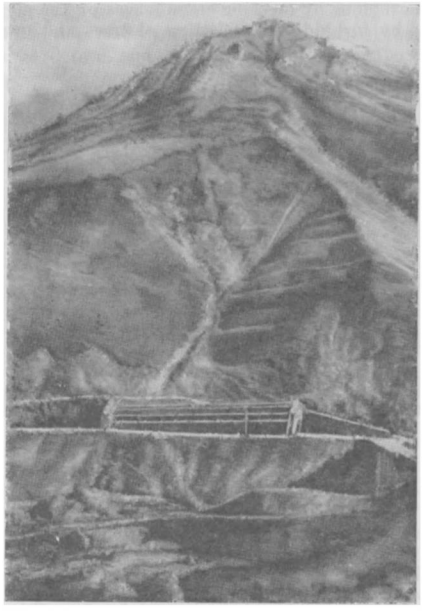


Fig. 216. Anti-avalanche gallery

the gallery roof, which is constructed over the road, without inducing impact loads. For this reason galleries are usually located on the bench of a cutting. The roof is covered with earth in order to continue the natural slope (Fig. 217). Galleries are constructed of stone or reinforced concrete.

When there are no snow fences, sometimes small avalanches are started artificially to clear away the snow. Such snow avalanches are created periodically to avoid the dangerous accumulation of snow.

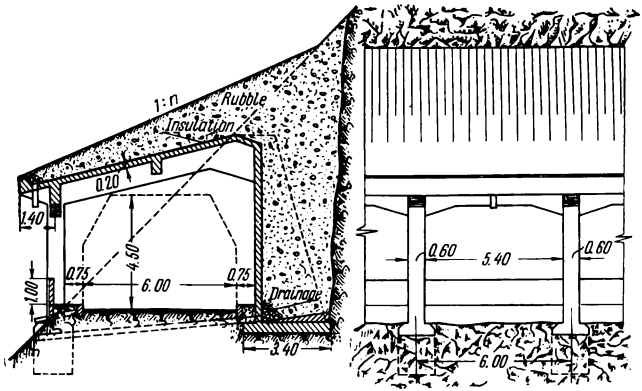


Fig. 217. Reinforced concrete frame gallery:
at left—cross-section; at right—longitudinal section

126. Features of Highway Design in Seismic Regions

When designing class I to IV highways in regions prone to earthquakes it is necessary to take into consideration the appearance of additional seismic forces, which may act in any direction. Usually the direction of these forces is assumed to be the least advantageous for a structure. Consequently, when carrying out a survey in seismic regions, the nature of the terrain should be taken into consideration, as well as geological and hydrogeological conditions under which the earthquake resistance of the road and its structures decreases.

Unfavourable conditions of land topography, from the point of view of earthquakes, include a highly broken relief such as ravines, precipitous slopes, canyons, slopes composed of weathered rock or broken by physical and geological processes, and lines of displacement (shear faults). Water-saturated macroporous (gravelly, sandy and clayey) soils are also seismically unfavourable, as are plastic fluid argillaceous soils.

The most favourable road locations are found in cemented rock and semi-rock formations, and in dense dry coarsely fragmental soils

The intensity of earthquakes in the region of road construction is assessed by a *seismic number*.

The members of structures, and also the stability of the roadbed in earthquake regions are designed with account taken of the seismic inertia forces, together with the dead and live loads of the structure. The wind load in this instance is not taken into account.

On checking the stability of the roadbed in seismic regions the value of the additional seismic force is determined from the formula

$$S = 1.5QK_s \tag{249}$$

where Q = vertical load, which under seismic action induces an inertia force (the own weight of a structure, soil, vehicles, etc.)

K_s = seismic factor depending on the design seismicity as follows

design seismic numbers	7	8	9
values of K_s	0.025	0.05	0.1

When designing retaining walls, in addition to the seismic forces of inertia, account is to be taken of the increase in the active pressure and the decrease in the passive pressure under the influence of the seismic action.

The active pressure of a loose soil with a rear vertical face of a retaining wall and a horizontal surface is calculated according to the formula

$$q_s = (1 + 2K_s \tan \rho)\, q \tag{250}$$

and the passive pressure under the same conditions is

$$q'_s = (1 - 2K_s \tan \rho)\, q' \tag{251}$$

where q = active pressure of soil without account of seismic action

q' = passive pressure of soil without account of seismic action

ρ = angle of internal friction of the soil.

The design seismicity for bridges and culverts is set depending on the class of the structure according to Table 43.

In regions with a seismic number of 8 or 9 it is not permissible to locate the road on slopes having gradients steeper than 1 : 1.5 unless special engineering and geological surveys have been undertaken, or the road is to be sited on rock.

The steepness of embankment and cutting slopes having an elevation of over 4 m in regions where the seismic number is 9 should be less than in normal conditions. In addition, the slopes of bridge abutments are also set at a lower gradient (Table 44).

In regions with a lower seismic number the sides of embankments and cuttings are given the same slope as in nonseismic areas.

When designing a roadbed in loose soils it is recommended to set the maximum height of embankments and the depth of cuttings for

TABLE 43

Name of structure	Design seismicity for seismic number at construction site			
	6	7	8	9
Large bridges on roads of classes I and II, expressways, urban roads and city arteries	7	8	9	—
Large bridges on roads of classes III and IV and district arteries	6	7	8	9
Medium-size bridges on roads of classes III and IV, district arteries, small bridges and culverts, retaining walls and wooden bridges on roads of all types	6	6	7	7

TABLE 44

Regions	Corresponding slope gradient						
	Embankments and cuttings				Bridge abutment cones		
Nonseismic	1 : 1.25	1 : 1.5	1 : 2	1 : 2.5	1 : 1.25	1 : 5	1 : 1.75
Seismic	1 : 1.5	1 : 1.7	1 : 2.2	1 : 2.5	1 : 1.5	1 : 1.75	1 : 2.0

regions with a seismic number of 8 equal to 15 m, and for regions having a seismic number of 9 to 12 m.

In cuttings, between the slope and the side ditch, berms should be constructed so that the weathered material will not block the roadbed. On hillsides having a gradient from 1 : 5 to 1 : 2 benches are made with a minimum width of 1.5 m at the foot of the embankments.

For the construction of retaining walls in earthquake regions the following rules are followed:

1. Walls of dry masonry should have a maximum height of 3 m and a maximum length of 50 m when the seismic number is 8 or less.

2. Walls of concrete and of cemented masonry constructed in regions with a seismic number of 8 should have a maximum height of 12 m, and with a number of 9 the figure is 10 m.

When constructing roads in earthquake regions special attention should be given to the introduction of anti-seismic measures.

127. Minor Structures in Mountain Regions

Highlands are notorious for the great quantity and intensity of rainfall. Cases are known when, during one shower, up to 15-20 per cent of the annual amount of precipitation was discharged. Appreciable longitudinal gradients and steep rocky slopes favour the building up of flows of great velocity and the formation of high water tables.

In summer, showers cause the extensive thawing of snow, and the resulting heavy flows lead to extensive washing out of beds and banks and to the deposition of sediment as alluvial fans. At the same time, during the dry period between rains, many thalwegs completely dry out, and this may give the surveyor a false impression concerning the size of the structure required in a given place. In establishing the opening of a structure the estimation of the discharge is a very complicated problem, owing to the still insufficient knowledge of runoff in mountain regions. The usual methods of determining the discharge (see Section 31) cannot be fully used since they do not take account of the nature of the relief, the steepness of the slopes, or the extent of the network of thalwegs and their tributaries in the catchment area, more especially when there are no data concerning the local conditions of storm intensity. Therefore, in the process of carrying out the survey, and apart from the data normally collected concerning the area and the slopes of the catchment area and thalweg, the high water level is established according to the visible traces of storm debris in the watercourses. At the same time the approximate velocity of flow can be estimated from the size of the boulders littering the stream beds. In the region through which the projected highway is to pass, local meteorological stations should be consulted for available data concerning the maximum intensity and duration of storms.

After rainstorms, mountain watercourses often carry shattered portions of trees, shrubbery, etc., and a great quantity of debris. The openings of small culverts and bridges may become blocked quite rapidly by this debris, necessitating frequent clearing out of the watercourses and culverts.

As a result, single-span bridges are to be preferred to multispan ones. According to sound engineering practice, the bridge openings should be not less than 3 to 4 m with a minimum vertical clearance of 1 m.

For periodical watercourses with stone beds and where no debris occurs, *percolating banks* are designed for the discharge of water, with a filtering arrangement against silting. Percolating banks are usually made when the flow does not exceed 10 m^3/sec. Sometimes these banks may be combined with a culvert capable of discharging 0.5 to 0,8 of the total flow.

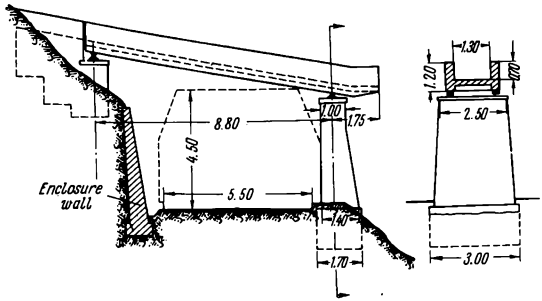


Fig. 218. Reinforced concrete elevated flume

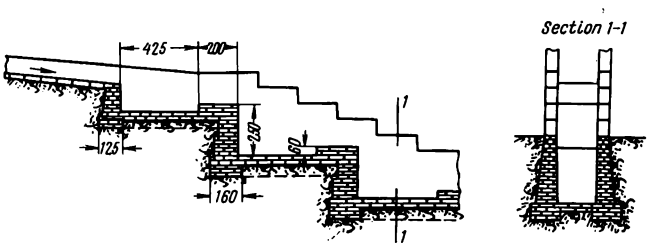


Fig. 219. Design of weir

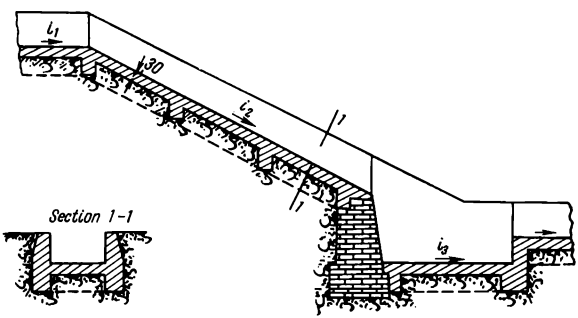


Fig. 220. Concrete chute

In highlands bridges and culverts are frequently located on road curves; in these cases the structure of the carriageway and of the roadbed is determined according to the general rules for the construction of roadbeds on curves.

128. Design of Approach Channels to Structures

The steep longitudinal gradients of watercourses and the hillside relief of the country greatly complicate the design of structures. In order to reduce the velocity of flow and the danger from erosion

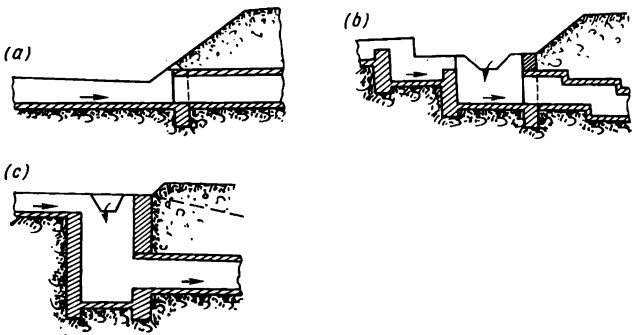


Fig. 221. Connection of approach channel with culvert: *a*—without stilling pool; *b*—with stilling pool; *c*—with deep pool

of the structure, special *approach channels* are built.

When designing hillside approach channels their type is chosen according to engineering and economic considerations. On precipitous slopes it is frequently expedient to allow the watercourse to pass over the road along a special *elevated flume* (Fig. 218). Weirs and chutes are more usual. Weirs may be constructed singly or in cascade with side retaining walls or without them, depending on the slope of the thalweg (Fig. 219). The *chute*, shown in Fig. 220, is a stone or concrete flume terminating in a stilling pool. With long approach channels, weirs and chutes can be combined and the chutes provided with a stepped or irregular bottom to reduce the flow velocity. The connection of approach channels with the opening discharging into a culvert can be designed in several ways, as shown in Fig. 221. The problems of designing the structure of the approach channels are analyzed in detail in special courses.

CHAPTER 21

ROAD DESIGN IN KARST REGIONS

129. Karst Processes

Karst processes occur in the body of massive soluble rock such as gypsum, limestone, rock salt and others. As a result of the combined action of ground and surface water such rock dissolves and is carried away by ground water. Thus, within the body of the earth caverns and cavities are formed, while on the surface of the earth all sorts of depressions are created, for example caves, sinkholes, hollows and furrows. The typical surface of such country, karst relief, is indicative of the development of karst processes, which should be taken into account when laying a road.

The construction of highways in karst regions is connected with the danger of their destruction by the continued development of karst processes, which depend on the extent of rock jointing, on the solubility of the rock in water, on the chemical composition of the ground water and on the topography.

According to Z. A. Makeyev, an indication of the intensity of karst process development can be obtained from the rate of formation of sinkholes over an area of 1 km² (Table 45).

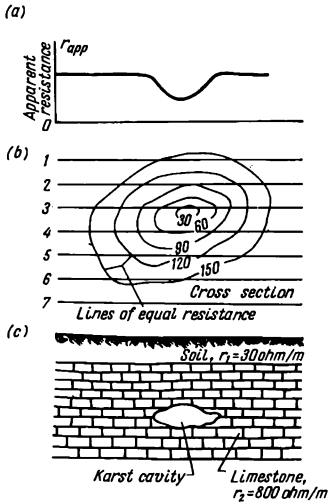


Fig. 222. Detection of karst cavity by electric resistivity survey:

a—alteration of electric resistance curve over a karst; b—contour chart of equal resistance lines enabling to determine the extension of the karst cavern; c—geologic section of the land

The age of the sinkholes can be determined according to the size and annular rings of trees growing in them, the extent of turf growth on the slopes, the rounding of the edges and the filling of the sinkholes with soil.

TABLE 45

Characteristic of topographical stability	Rate of formation of sinkholes per km²
Highly unstable	5-10 per year
Unstable	1-5 per year
Moderately stable	1 per 10-20 years
Stable	1 per 20-50 years
Highly stable	No new sinkholes recorded over the last 50 years

To determine the extent and the intensity of karst processes, and to find which stretches are unsuitable for road location, the geological stratification and the depth, composition and extent of jointing of the soluble rock should be studied during the survey. It will

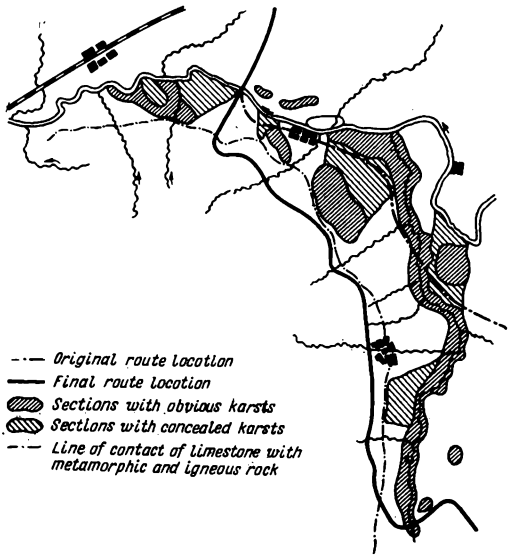


Fig. 223. Detailing of road layout in a karst region according to geological survey data

also be necessary to investigate the same properties of the soil mantle, as well as the relation between the locations of karst sinkholes and the geological character of the land. Information concerning groundwater conditions, the extent of water activity and the characteristics of the sources of water supply should be obtained.

To find underground karst cavities the electrical resistivity method of investigation can be used. This involves the measurement of the resistance of the rock body occurring at a definite depth. If there are caverns within the rock the electrical resistance changes abruptly at this point (Fig. 222) and the curve of the ratio of ground resistance to the distance between the electrodes is similarly distorted.

For a relatively safe route layout it is necessary that the poorly permeable soil mantle should have a minimum thickness of 8-10 m, the water-soluble rock layer should be of a small thickness and have insignificant jointing, the ground water should not be active and its flow restricted.

In a number of cases, as the result of a relatively small change in the alignment of the road, the danger of it being damaged by karst processes may be substantially reduced (Fig. 223). Ground subsidence at disused mines has a certain resemblance to karst phenomena. As a rule, such places are to be avoided, if practicable.

130. Design of Roads in Karst Regions

The control of karst processes is very costly and insufficiently effective, since for highway building it should have to be carried out over a very extensive area. Therefore, the unstable ground sections, where karst processes are active, should be bypassed as far as possible. If it is impossible to bypass a karst section, it will be good policy to align the route along watersheds or via high river terraces. At such elevated topographical locations the karst processes manifest themselves to a lesser degree than at the lower part of the slopes, where the rock is dissolved by a water supply which accumulates over a more extensive catchment area.

In regions where the karst processes have ceased, i.e., when no new sinkholes have appeared for many years, roads of high class may be constructed provided that suitable measures are taken to reduce water percolation into the soil within the limits of the road.

All special measures envisaged during road construction and the erection of road structures in karst regions are aimed at the single purpose of reducing the quantity of water penetrating into adjacent underground fissures and channels in the body of soluble rock. For this purpose the following measures should be taken:

(1) levelling the roadside, and draining off or diverting the water stagnating in topographical depressions. To prevent the percolation

of rain water adjacent to road structures, use should never be made of penstocks and bridges which are designed to accumulate water upstream of the structure. The beds of streams and of diversion ditches must be reinforced;

(2) filling of karst sinkholes with impermeable soil, the layers of which are thoroughly compacted. Deep borrow pits and soil quarries must not be located along the upper side of the embankment;

(3) clay grouting, cementing and asphalt grouting of fissures for reducing the permeability of soluble rock in the neighbourhood of road structures;

(4) the laying of a deep drainage system to intercept the ground water which flows towards the roadbed;

(5) filling the cavities and deep joints with sand and rubble in the neighbourhood of the roadbed, if these cannot be bypassed.

CHAPTER 22

DESIGN OF ROADS IN ARID REGIONS

Large areas of the southeastern part of the Soviet Union are covered by arid deserts and semi-deserts. According to Academician L. I. Prasolov, deserts and semi-deserts occupy approximately 10 per cent of the U.S.S.R. territory. Of these arid regions, 65 per cent are covered with grey soils, 25 per cent with sands and 10 per cent with saline soil.

The design and construction of roads in desert and semi-desert regions have their own features which depend on whether the route is being laid in irrigated districts, in saline soil or in loose sand.

131. Design of Roads in Irrigated Regions

At present it is considered the most advantageous to use irrigation systems with temporary irrigation channels instead of permanent ones. The basic method of irrigation is the overland gravity flow method of water supply, when a thin layer or stream of water flows over the soil and percolates into it. Temporary irrigation channels are built only for the period of watering and are levelled out prior to the commencement of other agricultural work.

A modern irrigation system consists of permanent channels, a temporary irrigation network and drain ditches.

The *permanent channels* of the irrigation system (Fig. 224) include:

(1) arterial channels which deliver the water from the supply sources to the delivery ducts and

(2) distribution channels which receive water from the arterial ones and distribute it between farms, as well as between separate irrigation sections within a farm. Depending on the proximity to the arterial channels there are distinguished distribution channels of the first order, second order, etc.

The *temporary irrigation channels* (network within a farm) include irrigation channels (a shallow irrigation network which is used for watering) and discharge and watering furrows for the uniform distribution of water over the plot.

The *water collecting and discharge network* serves for diverting the excess surface water from the irrigation network and from the watered plots.

The *drain system* is designed for controlling the level of the ground water discharging from the irrigated area.

Permanent channels serve large irrigated areas, which, in districts used for grain-growing reach 40 to 60 ha or more, and in cotton-growing districts equal 20 to 40 ha.

The road network in irrigated regions is coordinated with the irrigation system. As far as possible, the roads are located parallel to the channels. This enables more rational use to be made of the right-of-way and reduces the number of structures required at intersections with channels. The problem of road location in an irrigated region must be solved with account of the role played by the road in the total transport network, and it must be coordinated with the requirements of the agricultural organizations.

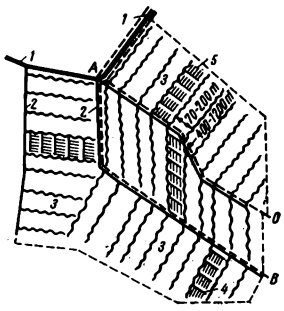


Fig. 224. Schematic view of an irrigation system:

1—arterial channel; 2—distributary channels; 3—temporary irrigation channels; 4—discharge furrows; 5—irrigation furrows

General-purpose highways, through roads which are the main arteries of the given district, are constructed in accordance with the usual engineering requirements. The location of these roads is coordinated, as far as possible, with the arterial channel network, while the distributory channels have to be adjusted where necessary to suit the road location.

Local roads serving the transportation requirements of farms, *link roads* between economic centres and railway stations or docks, and *secondary district roads* are all located along the shortest possible alignment adjacent to the distributary channels.

When building local roads for light traffic a certain departure from engineering standards is tolerated. It is the best policy to locate the route along watersheds and on land located above the irrigated fields. In flat country where efficient drainage is difficult, it is preferable to locate the route along operating open collecting drains. The minimum distance from the toe of the embankment to the edge of the collector should be 3 to 4 m. When it is necessary to lay a collecting discharge network along the channels, the distance between the edges of the channel and of the road drain trench should be at least 4 to 5 m.

When a road is laid along channels which are constantly filled with water, the roadbed will be in unfavourable conditions of exces-

sive moisture content, as a result of which the strength of the pavement will be reduced, and deformation and even destruction of the surfacing may occur. To avoid these effects the bottom of the roadbed along the road centre line of general-purpose roads should be sufficiently elevated either above the water level in the irrigation network, or above the ground-water table in the vicinity of channels, where, owing to the infiltration of water from the channels, the water table is higher than in the adjacent country.

The water conditions of the roadbed will be especially unfavourably influenced by the periodic leaching of fields in regions having a saline soil, which is usually carried out in spring. During this period from 5,000 to 15,000 cubic metres of water is used per hectare, which rapidly raises the ground-water table. Therefore, the basis for setting the elevation differences between the grade and the ground lines should be the high water level reached during the period of field leaching, which in some cases is 0.5 to 0.6 m from the surface of the ground.

The summer rise in the ground-water table during the period of cotton plant irrigation has no harmful effect on the roadbed soil owing to the intensive evaporation.

For roads having hard surfacings the elevation of the roadbed bottom along the road centre line above the sources of soil saturation should be taken the same as for a locality with prolonged surface water ponding or with a high ground-water table.

The elevation of the subgrade bottom above the winter-spring ground-water table, or above the water level in channels should be as follows:

	Elevation of subgrade bottom al ng road centre line, metres
Gravelly soils and sands	0.6-0.7
Fine silty sands and silty loam	0.7-0.9
Heavy and medium silty loam	1.3-1.5
Silty light loam and silty soil	1.7-2.0

On saline soil the elevation of the subgrade bottom is increased by 10 to 15 per cent, depending on the extent of salinity. This elevation should be 0.8 m above the earth surface in irrigated regions in the case of stretches with an assured runoff of water from the borrow pits, and 1.2 m on depressed sections where prolonged flooding of basins may occur, and on rice fields. On territory to be irrigated the bottom of the roadbed along the road centre line should be elevated to a minimum height of 1 m above ground level on nonsaline and slightly saline soils and to 1.2-1.5 m on medium and strongly saline soils, with a view to the future rising of the ground-water table after the commencement of irrigation.

Data on the water levels required for irrigation and the dimensions of channel elements are given in Table 46.

TABLE 46

Type of channel	Water level above channel or field below, cm	Raising of dyke level above water in channel, cm	Bank width, m
Temporary irrigation channel	Furrow irrigation 5-8		
	Controlled flood 6-12	10	0.3
	Flooding up to 20		
Distributary channel of second category	5-10	15	0.5-0.8
Distributary channel of first category and arterial channels having a flow of less than 2 m^3/sec	10-15	30	1.0-1.25
Ditto, but flow 2-5 m^3/sec	10-15	40	1.25
Ditto, 5-10 m^3/sec	10-15	40	1.5
Ditto, 10-20 m^3/sec	10-15	40	2.0

If it is impossible to provide the required elevation of the roadbed bottom then moisture barriers and layers stopping water capillary rise are arranged in the body of the roadbed.

The requirement of considerable elevation of the roadbed bottom contradicts, to some degree, the demand that maximum economy be observed in the use of valuable irrigated land for road construction. Therefore, it is good practice to coordinate the construction of a roadbed in irrigated regions with the general layout and preparation of the territory for irrigation, or, alternatively, special soil quarries will have to be provided on land not suitable for agricultural purposes. The surplus soil from cuttings should be spread over the depressed parts of the adjoining fields.

The profile of a road in irrigated country should be designed parallel to the profile of the channel, except for stretches where the channel is laid on high embankments or in deep cuttings. In irrigated zones the roadbed is located on embankments.

The ditch-basin and traverse drain cross-sections are the ones mainly used for general-purpose district and link roads.

The ditch-basin cross-section is used with a deep ground-water table. The discharge line is separated from the irrigation water by the erection of protective ridges 0.5 to 0.6 m high (Fig. 225*a*). If

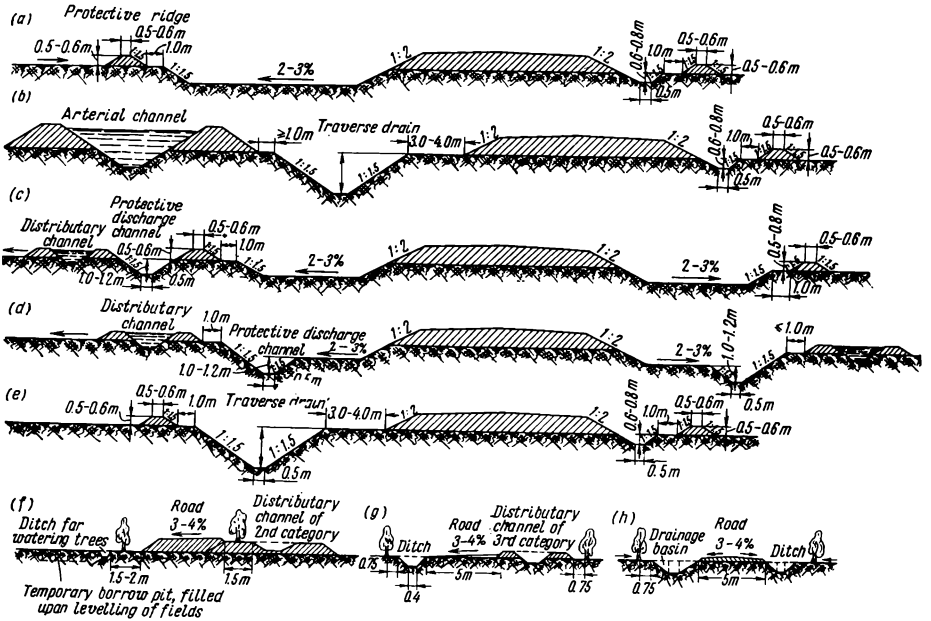


Fig. 225. Recommended roadbed cross-sections in irrigated regions:

a—ditch and borrow pit cross-section on irrigated land; *b*—ditto, with road located along arterial channel; *c* and *d*—ditto, along distributary channel; *e*—ditto, along a traverse drain (collector); *f*—cross-section of internal administrative road; *g* and *h*—cross-sections of field roads

the road is located along an arterial channel from which an appreciable quantity of water will percolate into the soil, a traverse drain is arranged between the road and the channel (Fig. 225*b*). To reduce the width of the right-of-way, one or both borrow pits may be partly backfilled after construction of the road by using the surplus earth from field levelling.

When the road is laid parallel to distributary channels, protective discharge channels are excavated along the field side of the borrow pit, or adjacent to its outer edge (Fig. 225*c* and *d*).

The transverse drain cross-section requires a smaller overall width, because the soil excavated for making the collector drain is used for filling the embankment. This road cross-section is used when the ground-water table is close to the surface and the road building and land reclamation work is carried out simultaneously. The protective berm between the embankment and the collector should have trees planted along it.

Local and field roads are located very close to the channels. The roadbed of a local road is arranged on a widened dyke of a second-category distributary channel (Fig. 225*f*). The borrow pits excavated between the road and the field are later backfilled when the fields are levelled and the channels are cleared.

Field roads are situated next to the channels (Fig. 225*g* and *h*). In the interests of road safety, between the road and the channel a berm is constructed which is also used for the dumping of deposits cleaned out from the channel.

Along the roads built parallel to channels, trees should be planted to decrease the evaporation of water from the channels. They intercept the percolating water with their root system and stabilize the slopes. Trees should be planted along the road on the berms and borrow pits in one to four rows. Fruit and mulberry trees should be widely used for such plantings. Special tracks may be necessary to give vehicles access to the adjoining fields.

Bridges for road crossings over distributary channels are built with a minimum vertical clearance over the water surface, since there will be no possibility of flooding in the channels. This clearance should not exceed the distance between the top of the dam and the water level in the channel.

132. Design of Roads in Saline Soils

A saline soil is defined as one containing not less than 0.3 per cent by weight of highly soluble salts, found within 1 m of the surface. Saline soils occur in arid regions as continuous beds and occasionally as separate stretches between nonsaline soils. Saline soils are divided into two groups: black alkali and white alkali soils.

Black alkali soils do not contain highly soluble salts in the upper soil horizons. Their physical and mechanical properties are determined by sodium ions in an absorbed condition. In the free state highly soluble salts are found in black alkali soil at a depth of over 50 cm.

Black alkali soils swell appreciably when wetted and are impervious. If an earth road is laid on black alkali soil, after a comparatively short rainfall it becomes difficult to drive on since the mud formed on the surface of the road sticks to the wheels and makes them spin. These soils dry out slowly.

Black alkali soils can be used for building roads with hard surfacing. However, the slopes of such embankments and cuttings are unstable and are prone to slip and flow. For this reason it is necessary to provide for slope and shoulder stabilization and for the thorough drainage of water from the roadbed.

White alkali soils contain in their upper layers, in a free state, more than 1 per cent of highly soluble salts, mainly as chlorides, sulphates and carbonates of sodium, calcium and magnesium.

Atmospheric precipitation, the intensity of which in arid regions is small, is capable of washing down only the most soluble of the salts, and as a result the greatest part of these accumulates in the upper horizons of the soil. In some cases the quantity of salts in the soils may be so great as to be precipitated as a layer on the surface of the soil (swollen saline soil—salt lake deposits containing an excess of sodium and magnesium sulphates).

White alkali soil is usually disseminated in small local areas amongst other desert and semi-desert soils and is found mainly in topographic depressions with a high level of stagnant saline ground water (minor depressions, troughs and lakes). On irrigated areas, on the contrary, spots of white alkali soil may be found on micro-elevations where the salts accumulate owing to capillary rise.

Prof. V. A. Kovda distinguishes four characteristic zones of salt accumulation in soils:

(1) Sulphate-carbonate typical for forest-steppe, where the salt content in the ground includes sodium carbonate Na_2CO_3, sodium sulphate Na_2SO_4 and sodium silicate Na_2SiO_3. The salt content in the upper horizons of saline soils reaches a maximum of 0.5-1.0 per cent. With a Cl'/SO_4'' ratio of less than 0.3 we have what is called sulphate salinization. Should the content of CO_3'' and HCO_3' ions in the soil exceed one third of the total content of Cl' and SO_4'' ions, we have soda salinization.

(2) Chloride-sulphate (steppe), where the sulphate content is greater than the chloride, the salts contained being Na_2SO_4 and NaCl $\left(\frac{Cl'}{SO_4''}\right.$ ranges from 1 to 0.3$\left.\right)$. The salt content in the upper layers is 2 to 3 per cent.

(3) Sulphate-chloride (semi-deserts), where chlorides are in excess of the sulphates, the soil containing NaCl, Na_2SO_4, $CaSO_4$ and $MgSO_4$ $\left(\frac{Cl'}{SO_4''}\right.$ varies from 1 to $\left.2\right)$. The salt content in the upper layers is 5 to 8 per cent.

(4) Chloride (deserts), characterized by a substantial predominance of chlorides over sulphates $\left(\frac{Cl'}{SO_4''} > 2\right)$. The soils contain the following salts: NaCl, $NaNO_3$, $MgCl_2$, $MgSO_4$, $CaSO_4$; in the upper layers the salt content may be as high as 15 to 25 per cent.

The content of soluble salts in the soil has a substantial influence on its physical and mechanical properties. When such soils are moistened, their resistance to external load decreases sharply, and during rainy periods the possible failure of slopes by slipping must be considered.

The salts contained in the soil may attack the road pavement. The pavement can be destroyed in a period of two to three years by magnesium and sodium sulphates if their content amounts to only 1 per cent. Weakly reactive chloride salts such as NaCl, $MgCl_2$ do not destroy the surfacing even when their content is over 5 per cent. Limestone surfacings are the most stable, but the same cannot be said about pavements made of igneous rock.

The destructive action of soluble salts on bitumen and tar manifests itself as leaching and emulsification of the binder. The most resistant surfacings are those built by hot application, using viscous bitumens. Cold-laid surfacings are less stable, particularly when cut-back bitumens are used.

The negative properties of saline soils noted above make it necessary to take action to prevent the penetration of salts into the roadbed from the base soils. However, owing to the difficulty of building the roadbed and pavement over saline soils, designers should endeavour to bypass individual sections where intensive salt deposits occur.

Sections having different degrees of salinity are characterized ecologically by particular plant species (saltworts).

If an embankment is made of saline soil, i.e., soils containing highly soluble salts, then, depending on the conditions of drainage and the embankment height, the soil may become stratified. If, however, the road crosses saline soils as a low embankment and the capillary rise of groundwater containing soluble salts into the roadbed is inevitable, then further salinization of the embankment soil is possible.

Since during the construction of the roadbed the soil is mixed, the extent of salinization, according to Prof. V. M. Bezruk, is defined by the average salt content in the top 1-metre layer of soil, within

which the borrow pits are excavated. Within this layer there will also take place the seasonal transference of salts (rise and leaching).

The maximum permissible salt content in roadbed soils depends on the nature of the salt. The chlorides NaCl, $CaCl_2$, $MgCl_2$ in small quantities (up to 3 per cent) improve soil stability; the roadbed becomes unstable only when it contains more than 8 to 10 per cent of these chlorides. The presence of from 2 to 5 per cent of the soluble sulphates Na_2SO_4, $MgSO_4$ has an adverse influence on soil consolidation since upon crystallization in the dry season they expand and loosen the roadbed.

The permissible salt content in roadbed soils is determined by the quantity which can be dissolved in the water filling the pores of the soil when the latter is compacted at the optimum moisture content. In this determination a correction is necessary to allow for the water film around the soil particles which will not dissolve the salts, and for the increase in the volume of sulphates upon their crystallization. A classification of saline soils according to their suitability for road construction is given in Table 47.

TABLE 47

Extent of soil salinity	Average salt content in upper 1-metre layer, %		Possibility of use in road construction	
	Chloride and sulphate chloride salinization	Sulphate and chloride sulphate and soda salinization	Use in roadbeds, etc.	Building of subgrades of soil stabilized with binders
Slight salinity	0.3-1	0.3-0.5	Suitable	Suitable
Medium salinity	1-5	0.5-2	Suitable	Suitable with certain limitations
High salinity	5-8	2-5	Suitable with certain limitations	Unsuitable
Excessive salinity	8	5	Unsuitable	Unsuitable

In arid regions where ground water occurs only at a great depth, the roadbed can be constructed of slightly saline soil according to the normal rules.

If there is a possibility of water seepage from outside, however, the embankment should be safeguarded by protective earth ridges.

In medium and highly saline soils it is necessary to ensure the thorough diversion of all water away from the roadbed.

When using saline and highly saline soils where the ground-water table is near the surface, it is necessary to take measures against a further increase in soil salinity in the embankment by salts carried up in the capillary water. For this reason the bottom of the roadbed along the centre line of the road should be sufficiently elevated above the level of the surface and ground-water table. This elevation should be greater than the one used in similar conditions for nonsaline soils (Table 48).

TABLE 48

Soils	Minimum elevation of roadbed bottom above ground-water table in winter and spring, metres	
	Slightly and medium saline soils	Highly saline soils
Medium- and fine-grained sands, light coarse-grained sandy loams	0.5	0.7
Silty sands, light sandy loams	0.9	1.1
Heavy loams, clays	1.4	1.6
Silty and heavy silty sandy loams, light silty and heavy silty loams	1.6	1.9

If drainage of the surface water is ensured and the ground water is found at a considerable depth, then the roadbed on medium- and high-saline soils is located on embankments, taking their height 20 per cent greater than that indicated in Table 14.

Should it be impossible to ensure the indicated elevation of the roadbed bottom in high-saline finely dispersed soils, then in embankments, at a depth of 65 to 75 cm from the pavement surface, a layer from 15 to 20 cm thick consisting of gravel 5 to 7 mm in size may be designed to hold down the capillary rise. To prevent pollution of this layer coarse-grained sand mats from 3 to 5 cm thick are located above and below it.

In the absence of soda salinization a 5- to 8-cm insulating layer of soil processed with viscous bitumen or tar may be designed.

On wet saline soil embankments must be constructed of imported soil, including soils naturally salinized, but within tolerable limits. It is recommended that the lower layers of embankments be constructed of sand or sandy loam carried up to a height exceeding the capillary rise.

133. Road Survey and Construction in Sandy Deserts

The features of climate and topography in sandy deserts substantially complicate the conditions of construction and operation of a road. In sandy deserts the topography is subject to continuous change, being governed by the intensity of the wind which sets the sand particles in motion. The following table relates wind velocity to the critical size of particles under which their motion begins.

	Particle diameter, mm	Wind velocity, m/sec
Finest desert sand	0.03	0.25
Very fine sand	0.12	1.50
Fine sand	0.32	4.00
Medium sand	0.60	7.40
Coarse sand	1.04	11.40

The movement of sand grains depends on the smoothness of the surface over which the sand is being transported. Level and smooth surfaces, e.g., flat clay desert land and dried-out salt lake beds, as well as thick sand deposits, offer little resistance to sand movement.

The flow of a stream of wind and sand over the irregularities of land relief is accompanied by local increases in wind velocity, eddies and stagnation zones. In eddy zones the sand is carried into aerial suspension, and in stagnation zones it is deposited.

The taking up and deposition of sand grains by the wind creates a general movement of surface sand layers, in the form of small ridges. The sand grains are pushed up the slopes of the ridges, and, after having been carried over the summit, they fall and are deposited in the stagnation zone on the leeward side. As a result, the sand hills steadily move in the same direction as the wind. Such sands are called *drifting sands*. The speed at which the sand ridges or dunes travel decreases as their height increases.

The following characteristic topographic forms of sand deserts, which are created by wind action, are distinguished: barkhan sands, barkhan chains, sand ridges and hummocky sands. The formation of each of these topographic forms is connected with specific conditions of sand displacement and the direction of the prevailing winds.

Barkhan sands (Fig. 226) consist of sand hills, crescent-shaped in plan, with their arms orientated in the direction of the wind. The flatter, windward slope has an inclination of 5 to 12 degrees, and the steep leeward one—28 to 36 degrees. This topographic form is the least stable and readily moves under the action of the wind. Single barkhans will appear on the boundaries of loose sands and on level, denuded and flat clay and saline surfaces where there may be only a comparatively small quantity of sand inflow.

In areas where the prevailing winds change their main direction twice a year (for example, they blow in one direction in winter and in the opposite one in summer), in drifting sand masses there

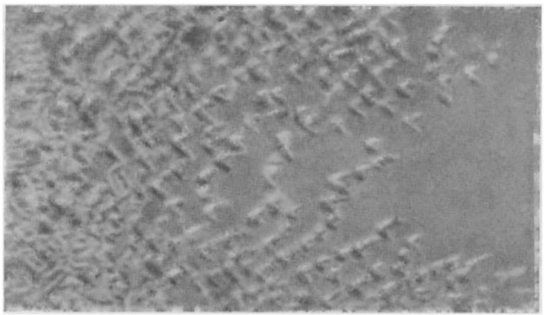


Fig. 226. Air photograph of barkhan sands

Fig. 227. Air photograph of barkhan chains

are formed *barkhan chains* (Fig. 227), which are arranged at right angles to the wind direction. These chains are asymmetrical ridges from several to over 200 metres in height.

When the prevailing winds have a constant direction throughout the year, *sand ridges* are formed (Fig. 228) which are stretched out in the direction of the wind. The ridges are spaced almost equidis-

tantly (on an average 180 m). The formation of the sand ridges is caused by sand being blown out by air eddies whose horizontal axes correspond to the direction of the depressions.

Hummocky sands comprise small sand hillocks of irregular configuration, anchored by vegetation. The height of the sand mounds does not exceed 6 to 8 m. The steepness of their slopes is approximately equal on all sides.

To determine the topography of loose sands during surveys, it is good to employ aerial photographs and air surveys.

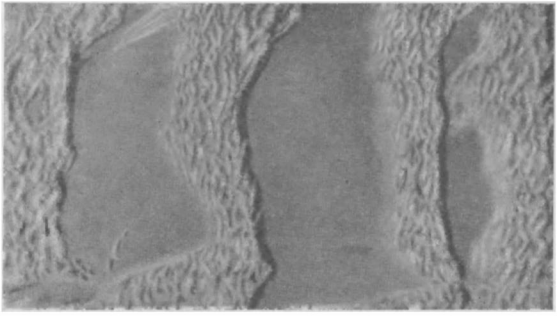


Fig. 228. Air photograph of sand ridges

The quantity of sand transported depends on the power of the wind, which is proportional to the square of its velocity. When assessing the conditions of sand movement, therefore, it is most helpful to make out a chart in the form of a *wind-velocity rose*, or *dynamic wind rose*. When plotting dynamic wind roses, along each bearing is traced a vector comprising the sum of the products of the wind velocity squares and the frequency of their occurrence. For plotting a dynamic wind rose, only periods when the sand is in motion are considered, while the time when the sand is temporarily stabilized by moisture or covered with snow is excluded. Dynamic wind roses are used to work out measures necessary to protect the road from being covered by drifting sand (Fig. 229).

Sand movement may involve the following conditions:

(1) forward motion, when during the year the winds from one quarter greatly exceed the strength and frequency of winds from all other quarters;

(2) an alternating pattern of winds where the intensity and persistence of summer and winter winds are approximately equal and the chains of barkhans periodically alter their configuration, but remain essentially in the same place;

(3) a combined forward and alternating motion, when the chains of barkhans move generally in one direction but with periodic retreats, and the speed of sand movement in this case is less than in case (1) above. The pattern of sand displacement in the deserts of the U.S.S.R. has been thoroughly studied, and special maps of sand movement are available.

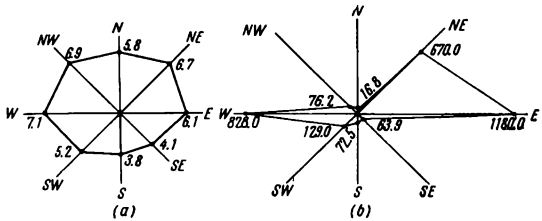


Fig. 229. Wind rose:
a—normal; b—dynamic

When preparing a programme for the organization of road construction work in sandy deserts, and also when developing the operation service, it is necessary to take account of the distinctly continental climate, the scarcity of water and the low density of population. Here, since there are no local roads, it is very difficult to plan the organization of the work. In practice it is not always possible to undertake construction work simultaneously over the whole route, since temporary roads have first to be built to convey materials, water, etc., to the site.

During road construction it is necessary to ensure normal conditions of work for the men and machinery. Loose sandy soil tends to bog down scrapers and other road machines, and requires the use of more powerful tractors than with other soils. The amount of soil which can be carried in scraper buckets is reduced, as is the volume of earth rolled by the blade of a bulldozer. The dust in the air causes increased wear of mated parts of road machinery and vehicles. The wear of internal combustion engines increases substantially because of poor functioning of the air filters and the cooling system, if these are not properly adapted for operation in hot climates. The high ambient temperature requires action to be taken to protect the workers from the sun, such as the fitting onto road

machinery of cabs, which must be painted in light colours, the use of protective awnings, etc.

When locating roads in sandy deserts designers should, as far as possible, avoid sections where the sand is loose, locate the route along sections of sands anchored by vegetation and having a smooth relief, and bypass open sands which are clearly unstable. Preference should be given, where possible, to districts consisting of coarse sands. In areas of sand ridges or dunes it is preferable to locate the route along the ridges or, alternatively, along the spaces between them. The alignment of the route over the slopes of sandy sections of the topography always involves the necessity of introducing complex stabilization work.

Cuttings should not be constructed in these areas. In some cases it may be more expedient to design a road on an embankment rising along the slopes of a hummock, observing the permissible gradients. Roads should never be located in a zone of sand deposition.

It will be good practice to site the road away from ridges and barkhans to a minimum distance equal to twice their height. By observation of the wind rose it is possible to establish on which side of the ridge it should be safer to lay the road in their proximity.

Roadbeds, including ones on clayey soils, are designed on embankments with an elevation of about 50 cm above ground level and with gentle slopes of 1:4 to 1:5.

Owing to the high water permeability of sand, drains on sand sections are not necessary. Drainage is provided only on sections with clayey soil and also on route stretches near the sand border line, where the water runs off during rainfall from bare hummocks covered with impermeable rock. Since sand is easily eroded by water one should not allow water to flow alongside the road for any appreciable distance.

The construction of a highway roadbed alters the local conditions of wind-blown sand streams. In the stagnation zones which are formed next to the road, sand deposits will occur, and in places where the wind eddies, the sand which has been used for the road embankment will be dispersed. For this reason, the choice of a rational embankment configuration means finding a solution to the aerodynamic problem involved in obtaining the best streamlined cross-section.

In the Soviet Union a method has been put forward for the free transfer of sand over the road, which takes account of the transfer of sand occurring in practice over stretches of clayey soils in a desert. Basically this method consists in creating along the roadside strip such conditions for the movement of the wind-blown sand stream that the formation of deposits will be impossible. For this purpose, within the limits of 40 to 50 m from the road centre line, all topo-

graphic irregularities which are capable of retaining the sand must be levelled out on the roadside. The road embankment is given a streamlined cross-section with gentle slopes of 1:3 to 1:5 and with rounded roadbed edges (Fig. 230). Such a section easily blends in with the surrounding topography. The shoulders are given a slope not exceeding 4 to 6 per cent. To prevent the erosion of sand the embankment slopes are stabilized with a layer of cohesive soil or gravel, at least 15 cm thick. If there is no clayey soil, the slopes

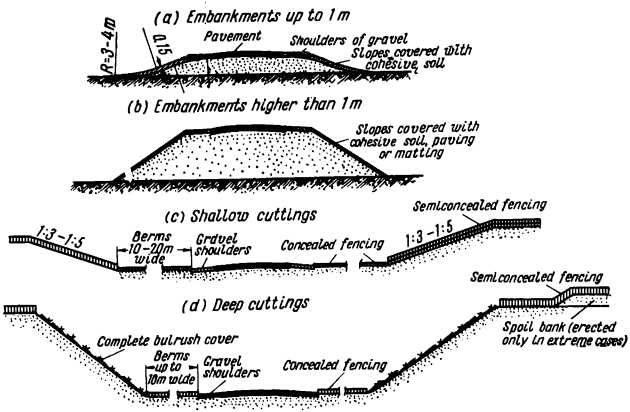


Fig. 230. Road cross-section in mobile sand zone

can be stabilized by binding the sand with cutback bitumen or a bitumen emulsion. The shoulders are reinforced with gravel processed with a binder.

To improve the conditions of sand transference, at a distance of 30 to 40 m from the road a cut-off ridge 1.5 to 2 m high is built, which is stabilized with emulsion against dispersion by the wind. The flow of the wind current over the cut-off ridge creates eddies and ascending currents which carry up into the air the sand which was previously transported by the wind along the surface, and therefore facilitates its transportation over the road.

High embankments can be built with a normal cross-section with side slopes of 1:1.5. Depending on local conditions, the slopes should be stabilized with soil treated with binders, with tessellated paving, with continuous paving or with screens made of branches of trees, shrubbery, etc.

The problem of a rational cross-section for cuttings is very complicated. With a certain optimum ratio of the cutting width at the top to the depth of the cutting, winds of sufficient force will generate eddy currents inside contributing to the blowing away of the sand falling into it. However, with weaker winds the falling sand will remain in the cutting.

An improvement in the self-cleaning properties of cuttings by using gentler slopes or by giving them convex streamlined contours cannot completely eliminate the danger of sand drifts, although it somewhat betters the conditions of road operation. The slopes of cuttings and the adjacent stretches of the right-of-way are fastened with covering screens. Instead of dumping the excavated sand into spoil banks, it should be hauled some distance away and levelled out on the leeward side of the right-of-way.

It is impossible to eliminate completely the deposition of sand in cuttings. Therefore, the roadbed in cuttings is often widened. Sand will be deposited during high winds within the limits of the additional road width, without interrupting traffic.

Insufficient knowledge of the formation of sand deposits on roads makes it necessary to increase the staff of the road operation services and to provide means for clearing the road from drifts.

Borrow pits, as a rule, are soon covered over with sand. In drifting sand areas, they serve initially to give some protection to the embankment, since the transported sand is deposited in them. In districts where the upper sand crust has been stabilized, the sides of borrow pits may be blown off by the wind and become the cause of drifts on the roads or even of complete road destruction. In these cases the borrow pits should be located on the leeward side of the road and their beds should be stabilized with buried brushwood.

134. Sand Stabilization

The streamlining of the roadbed cross-section cannot, in itself, fully protect the road from drifts. It is also necessary to stabilize the sand stretches flanking the road.

A quite reliable and lasting road protection from sand drifts is given by the parallel planting of a wide grass or shrubbery border. This method has been proved by experience on a number of sections of the Ashkhabad and Astrakhan railways. However, several years are required in order to produce an effective roadside protective planting. In many cases as, for instance, with highly saline and highly mobile sands, where ground water is found only at a considerable depth, successful methods for sand stabilization by vegetation have not so far been produced.

Therefore, simultaneously with sand stabilization by vegetation the road needs to be protected by fences. The action of such a protection consists in creating regions of calmer air adjacent to the fences, where the sand is deposited.

Observations have shown that the contours of the sand dunes in close proximity to the protected sections of a road depend on the degree of wind penetration through the erected fences.

Adjacent to *continuous fences* the deposits accumulate mainly ahead of the fence. Where the sand has piled up level with the top

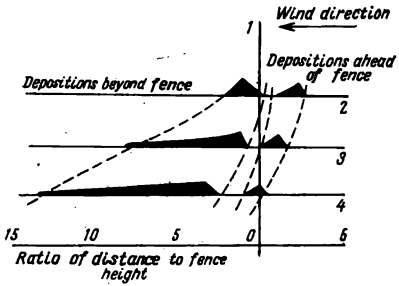


Fig. 231. Relation between sand deposition and density of fencing:
1—line of fence erection; 2—continuous fence; 3—fence with 25% of openings; 4—fence with 50% of openings

of the fence the deposits have a triangular cross-section with the slopes inclined at a gradient equal to the angle of internal friction of the sand.

When a road is protected with *lattice fences*, sand is blown by the wind through the fence. However, here the wind velocity is reduced and the sand will be deposited directly beyond the fence. The length of sand deposits will grow with an increase in the relative proportion of gaps in the total length of fence. Lattice fences give gently sloping deposits, uniformly distributed over the diversion strip (Fig. 231). Such fences can be easily transferred to a new location after they have become overtopped. Lattice fences are the most expedient for retaining sand which is being blown towards the road.

If the route is located at an acute angle to the direction of the prevailing winds, instead of retaining the drifting sand it is possible to divert the sand stream parallel to the road by the erection of continuous *reflecting fences*.

By properly combining the direction of the reflecting fences with that of the road it is possible, in principle, to arrange for the sand to pass over the road, selecting for this purpose a stretch which follows the ground line and designing a smooth surface of the shoulders and slopes.

For stabilizing the elements of the terrain in the vicinity of the road continuous fences of various types are used (Fig. 232).

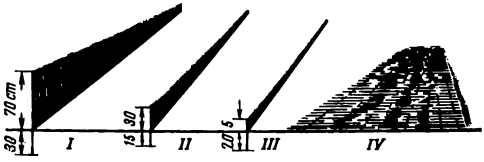


Fig. 232. Various types of continuous fences:
I—high; *II*—semi-concealed; *III*—concealed; *IV*—covering screen

The topographic profiles are stabilized by erecting a series of high, semi-concealed and concealed fences. A covering screen is used for fastening the roadbed.

For *artificially altering the sand topography* along the roadside, the power of the wind can be utilized by installing protective fence lines that partially arrest the movement of the sand and partially divert it in the desired direction.

By installing fences it is possible to slow down the movement of sand dunes, level out the barkhan topography, increase the interval between dunes along the road (in zones where the wind changes during the year) or ensure the accumulation of sand ridges.

Single barkhans, moving towards the road, can be transferred over the road at a low velocity by stabilizing the lower part of the barkhan with latticed and embedded fences (Fig. 233), so that the sand being transferred will not impede road operation. When the top part of the barkhan has been blown over the road the fastening is taken off, in order to open up and permit the blowing over of new sand layers. The extent of the open part of the barkhan surface can be established only by experiment directly on the site.

The stabilizing effect of vegetation manifests itself by a reduction in the velocity of the wind-blown sand stream by the plants' stems, by anchoring of the sand with its highly developed root system, and by the gradual development of soil cohesion due to the accumulation of a vegetable soil composed of decaying organic matter.

Drifting sands have a comparatively uniform granulometric composition (single-size grains). The amount of available nutrient contained in them is small. In addition the sand contains salts

that are harmful to vegetation (chlorides and sodium sulphate). Only local species of vegetation, which have adapted themselves to growth in arid desert climate and in unfavourable soil conditions, can thrive in the sands. This is why the species of vegetation for the stabilization of a sand area should be selected by a land reclamation specialist.

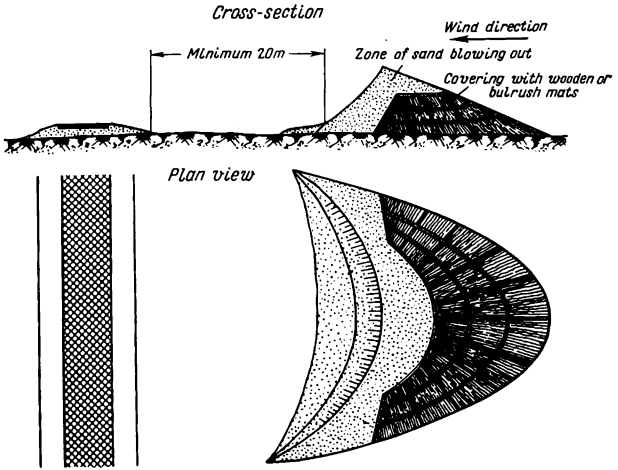


Fig. 233. Control of barkhan transfer speed by stabilization of base

In natural conditions, the process of sand stabilization by vegetation requires a lengthy period of time extending over decades. Active interference by man to promote sand stabilization can substantially accelerate this process. The artificial stabilization of sand with vegetation consists in sowing grass and planting cuttings of shrubs—sand stabilizers—with simultaneous fastening of the sand topography by means of fences. Good results are obtained by the use of bitumen emulsion for sand stabilization after grass has been sown.

In zones where the prevailing wind blows during one season in one direction and during another season in the opposite direction, the installation of fences on both sides of the road alternately results in the shifting of the dunes from the vicinity of the road.

When the season of the change in wind direction approaches, the fences are transferred to the opposite side of the road. This prevents the return movement of the barkhans which have moved away from the road. On the other hand, barkhans that were previously stabilized will start to migrate away from the road.

The above methods of fence employment for retaining the sand and for levelling the topography have one common drawback—excessive labour requirements. The operation staff has to effectively supervise the functioning of fence line installations and take into account in good time the influence of local conditions. Owing to this, all such protection has a temporary character. The planting of vegetation completes the group of measures taken for sand stabilization.

PART VII

Urban Streets and Roads

CHAPTER 23

DESIGN OF URBAN STREETS

135. Street Layout and Elements

The *plan of an urban street network* is determined by the type of town layout, the distribution of places of employment, of residential areas, of public buildings, stations, docks, and also by junctions with rural roads.

The layout of the older towns grew historically under the influence of social, topographic and climatic conditions. New towns and cities in the U.S.S.R. have been laid out according to a single plan adapted to the main requirement, namely, to provide the maximum facilities for the inhabitants. In this connection all efforts are made to distribute rationally residential quarters, industrial enterprises, railways and highways, green belts, taking into consideration natural conditions and, in particular, the land topography. The development and reconstruction of existing towns and cities are also based on a comprehensive study of the distribution of industry, transport communications, the selection of the most suitable districts for residential quarters and the creation of green belts.

The basic types of town layout can be classified as follows: *radial*, *spider web*, *rectangular* and *combination* (Fig. 234).

The majority of ancient towns (walled towns or fortresses) are characterized by the spider web layout. An example of such a layout is Moscow. In towns and cities located on the banks of a river or on a coast the layout may be an incomplete spider web system. Towns founded in the 17th and 18th centuries have, as a rule, a rectangular layout.

The combination layout, which combines the rectangular one with a series of diagonal streets, breaks the monotony of the rectan-

gular layout and results in the creation of beautiful squares and street vistas. An unsurpassed example of a city with a combination layout is Leningrad, which has become a school for Russian town planners and architects.

The street layout exercises a substantial influence on vehicle operation. In comparison with the shortest alignment (a bee line), the rectangular layout extends the average distance between points by road by 27 per cent, and the spider web layout by 10 per cent.

A street network is usually characterized by its total extent and density, i.e., the total street length in kilometres contained in one square kilometre of urban area.

As a result of the continued growth and development of urban areas, the urban street network develops, absorbing country roads which become urban streets. For this reason the street network must be laid out with a view to the disposition of points of traffic origin and to the arrangement of the road network in the suburban areas. Each street is designed in relation to its location and to its importance in the general street network plan.

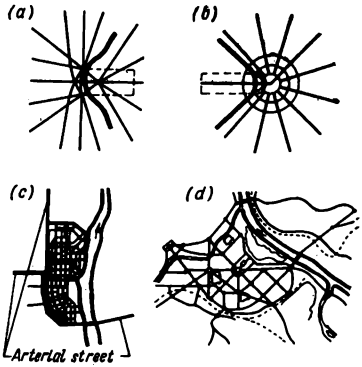


Fig. 234. Types of town layout:
a—radial; b—spider web; c—rectangular; d—combination

According to planning standards cities and towns are classified as important, with a population of 500 thousand and over; large, with 100 to 500 thousand inhabitants; medium, with 50 to 100 thousand inhabitants; and small, with a population up to 50 thousand.

The *classification of urban streets* is worked out according to their main functions, the character of buildings, the anticipated intensity and type of traffic, the extent of development of underground installations, the situation of the street in the street network plan and in relation to the rural highway approaches. Table 49 gives the classification used in the standards for urban street designing in the U.S.S.R.

The overall width of arterial streets of general urban importance is usually within the range of 30 to 50 m, and of arterial streets of district importance—25 to 35 m. The width of residential district

TABLE 49

Streets and roads	Principal designation
Expressways	High-speed communication between remote city districts, with large industrial areas beyond city limits and with rural highways of high type. Designed for intense vehicular traffic with grade separation
Arterial streets: (a) city arteries	Communication between residential, industrial and business districts and also with the town centre, with places of general importance (stations, parks, stadiums, freight yards, etc.), as well as with highways with traffic separation in one or different grades
(b) district arteries	Local communication within the limits of residential and industrial districts, communication between these districts and city arteries and expressways
Streets and roads for local traffic:	
(a) in residential districts	Transport and pedestrian communication between residential districts and arterial streets
(b) in industrial and warehouse districts	Transport and pedestrian communication between industrial enterprises, warehouses and arterial streets and through-roads
(c) access roads	Communication within city districts and with streets for local traffic; access to separate industrial enterprises
Sidewalks	Communication for pedestrians between residential areas and industrial enterprises, places of recreation, public centres, cultural and welfare centres, public transport stops. Lanes in parks

streets is selected in accordance with the height of the buildings along the street: with multi-storey buildings it is 25 to 30 m, and with low buildings or estate dwellings it is 14 to 20 m. When boulevards are set out the width of the street is increased by the width of the planted strips.

For small villages and settlements, in view of the small traffic intensity, the width of the streets and the type of carriageway may

be of somewhat lower standards. The width of district and village arterial roads is usually 25 to 35 m.

Rural roads, particularly those of inferior classes, often pass through inhabited localities and need to comply with the requirements of highways of the corresponding class, in addition to those of urban and village streets. This often creates substantial difficulties in the design, construction and operation of such roads. The width of roadways located within industrial enterprises is selected to facilitate the most compact location of the roads, sidewalks, underground and overground communication lines and of plantings, as follows:

	Width, m
Arterial roads for large industrial enterprises occupying an area of over 100 ha	32-40
Arterial roads for enterprises occupying an area of 50-100 ha	26-32
Arterial roads for enterprises occupying an area of less than 50 ha	20-26
Roads between blocks	10-20

The minimum width of a thoroughfare must be not less than the interval required by fire and sanitary regulations. The width of streets for local traffic is determined by the character and type of buildings. Thus, for instance, in a zone of multi-storey buildings (up to five storeys inclusive) the width should be within 25 to 30 m. In a zone of low buildings the width should be 14 to 20 m, and in a zone of estate buildings it may be 12 to 18 m.

The *elements of an urban street* include the carriageway, tramway bed, sidewalks, planted strips and cycle tracks. The carriageway width is chosen with a view to the estimated traffic intensity at peak hours and the respective traffic capacity of a single traffic lane, which is determined according to the street category, the distance between street intersections and the capacity of the latter. The width of a traffic lane for high-speed passenger cars and public transport vehicles is taken equal to 3.75 m for a design speed of 100 km/hr and above, 3.5 m when designed for trolleybus and autobus services, 3 m when passenger cars predominate, 2.75 m with two-lane roadways for single vehicles.

The traffic capacity of a single traffic lane between intersections is determined according to the same formula used for calculating the traffic capacity of rural roads. The traffic speed and the design values of the coefficients contained in the formula are selected depending on the expected traffic conditions. In urban conditions the traffic capacity of a street depends mainly on the traffic capacity of the intersections.

Traffic delays occur at street intersections because drivers must reduce the speed of their vehicles at traffic lights, stop, and then, starting from rest, steadily accelerate the vehicles to their normal speeds.

The reduction in capacity of a street is taken into account by a factor α which is determined according to the formula

$$\alpha = \frac{L}{L + v\Delta + \frac{v^2}{2}\left(\frac{1}{a} + \frac{1}{b}\right)} \tag{252}$$

where L = distance between intersections, m
v = traffic speed, m/sec
Δ = lost time of vehicle at red light, sec
a = average acceleration when starting from rest, m/sec^2
b = average deceleration when braking, m/sec^2.

Thus, the traffic capacity of a street, taking into account intersections, is given by the formula

$$N_s = \alpha N \tag{253}$$

where N = traffic capacity between intersections.

The value of the factor α depends mainly on the distance between street intersections and the traffic speeds. The factor α decreases with an increase in traffic speed and a reduction of the distance between intersections. According to Prof. A. Y. Stramentov, with a traffic speed of 40 to 60 km/hr and a distance between intersections of 300 m, the factor α is reduced to 0.4-0.5. With several traffic lanes in each direction the traffic capacity of each inner lane decreases in comparison with that of the outer lane as follows:

Capacity of the outer (first) lane	1
Ditto, second lane	0.85
Ditto, third lane	0.7
Ditto, fourth and following lanes	0.5

A considerable increase in the capacity of a road will be achieved by marking out the carriageway into lanes by traffic directions, while the division of the road into two carriageways (dual-carriageway) by a median is even better. On streets used by public vehicles and along which public service institutions are situated, the carriageway should be widened to provide for the parking of vehicles.

For preliminary calculations the traffic capacity of a single carriageway lane may be taken as indicated in Table 50.

With mixed urban traffic the different kinds of vehicles are reduced to an equivalent number of standard passenger cars, using the following reduction factors:

Passenger cars	1
Trucks with capacity up to 3 tons	1.5
Ditto, from 3 to 5 tons	2
Ditto over 5 tons, buses, trolleybuses	3
Combination vehicles, articulated trolleybuses	4
Motorcycles	0.5
Bicycles	0.3

TABLE 50

Type of vehicle	Maximum hourly volume with uniform traffic	
	Without intersections at grade	With intersections at grade
Passenger cars	1,000-1,500	500
Trucks (1.5-3 t)	800-1,000	350
Trucks (3-5 t)	600-800	350
Buses	200-300	100-150
Trolleybuses	100-130	60-90

The street carriageway can be designed as a common one for all means of transport, or it may have separate lanes reserved for one particular type of transport.

The carriageway curbs are situated at a maximum distance of 25 m from the building lines, or in such a manner that between the building line and the carriageway a uniform strip having a minimum width of 6 m is reserved for the passage of fire engines at a minimum distance of 5 m from the buildings. At the end of blind alleys, turning circles of 10 m radius or squares of 12 × 12 m should be constructed. The minimum widths of street carriageways are given in Table 51.

TABLE 51

Streets and roads	Width of lane, m	Minimum number of lanes
Expressways	3.75	4
City arteries	3.5-3.75	4
District arteries	3.5	4
Streets and roads for local traffic:		
in residential areas	3	2
in industrial and warehouse districts	3.5	2
access roads	3.5-2.75	1-2
squares	3.5	4
Sidewalks	total width at least 3 m	—

Note: For urban expressways the width of a traffic lane is in the range of 3.5-4.0 m, the usual number of lanes being four.

With a low traffic flow and a two-way trolleybus service the carriageway should have a minimum width of 10.5 m.

The *width of sidewalks* is selected according to the street category, the nature of the buildings and the number of pedestrians. It is assumed that a pedestrian in motion occupies a lane 0.75 m wide. When determining the width of the sidewalk designers should take into account the arrangement of shops and public service institutions, and also the possibility of public promenades along central streets, river embankments, etc.

The capacity of one sidewalk lane is 1,000 pedestrians an hour. The minimum width of a sidewalk is 1.5 m and only for streets with private housing may it be decreased to 1 m. In the vicinity of stations, theatres and underground stations the sidewalks should be widened by setting back the building line.

The sidewalks can be arranged differently depending on the total width of the street: next to the carriageway; between planted strips which separate the sidewalk from the carriageway and from the line of buildings; next to the building line, but separated from the carriageway by a planted strip, etc. In some cases a cycle track may be designed between the sidewalk and the planted strip.

Tramway and trolleybus pylons and street lighting and communication masts are installed on sidewalks at a distance of 0.35 to 0.50 m from its edge. In this case the width of the sidewalk is increased by 0.5 to 1.0 m.

Of great importance for decorating streets and improving their sanitary condition are *planted strips*. In addition, the planting of shrubs on medians, etc., is conducive to traffic safety. When deciding on the type of plantings the total width of the street, that of the sidewalk, the location of underground facilities and of tramway lines are taken into account. In northern towns trees are planted at a distance of 5 to 6 m from buildings so as not to cast too heavy a shade over windows, while in southern towns, on the contrary, the trees are brought nearer to the houses to provide shade and are planted at a distance of 2 to 3 m from buildings. Trees and shrubbery should be planted at the following distances from the edges of sidewalks: trees—minimum 1 m, shrubbery—0.5 m. The trunks of trees should be at least 0.5 m from underground cables and 2 m from gas pipes. When planting trees the positions of lamp-posts, tramway pylons and of power and telephone lines should be taken into account, so that a minimum of 1 m will remain between the tree tops and the wires. At street and square intersections plantings should be arranged so as to ensure proper visibility and traffic convenience. On wide streets where extra space has been provided, the surplus widths may be turfed. At a future date these strips may be used for widening the carriageway, sidewalks or for accommodating

a tramway. The minimum width of strips with plantings should be as follows:

	Minimum width, m
Single-row tree planting	2.0
Double-row tree planting	5.0
A strip of low shrubbery	0.8
A strip of medium shrubbery	1.0
A strip of high shrubbery	1.2
Grassed parking	1.0
Front garden	4-6

Cycle paths are provided on wide streets and are situated between the carriageway and the sidewalk, on both sides of the street. It is preferable that the cycle traffic be separated from the carriageway by a strip of grass.

The *width of the cycle path* is usually 1.5 m for a single path and 2.5 m for a double path. In populated areas cycles paths are designed for one-way traffic. They are located on both sides of the street, as a rule, between the roadway and the sidewalk, and are separated on both sides with strips of green parking at least 0.8 m wide. Two-way paths, 3.75 m wide, with a paint-marked centre line may be used only in parks and rural districts.

Tramway tracks are located down the middle of the street, or at one side of the street, or on both sides of a boulevard (one track on each side). The tramway track can be designed on the same level as the carriageway or on a separate bed of the railway type. The construction of a separate bed gives a substantial increase in tramcar speed and improves traffic safety. The cost of construction and operation of a separate tramway track is less than that when the tracks are arranged on the same level as the carriageway. However, such a track is difficult to cross, and it cannot be used for other types of transport. It is permissible to construct a separate bed on streets where the carriageway for each traffic direction is at least 6 m wide, and where the tramway will be crossed by other traffic streams at minimum intervals of 300 m.

The width of the tramway track at road level, with central suspension of the contact wires, is 6.95 m (Fig. 235*a*), and with lateral suspension of the contact wires 6.6 m (Fig. 265*b*). When a separate track is constructed its width may be correspondingly 7.35 and 7 m.

To protect the asphalt concrete surfacing from destruction by vibration of the tramway rails, these are separated from the carriageway by wooden blocks or cobblestone paving 0.4 to 0.5 m wide. A more modern type of tramway track design is on a concrete foundation with rigid anchoring of rails, which eliminates damage by vibration; this is shown in Fig. 236.

When determining the width of the street carriageway the clearance between a tramcar and an automobile body is taken equal to 0.4 m. The minimum distance from the track centre line to a sidewalk is 2 m.

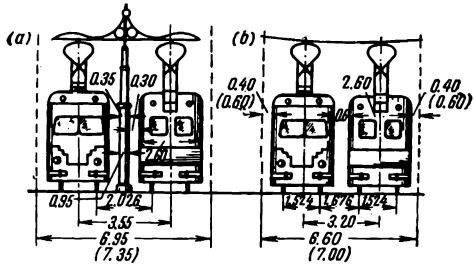


Fig. 235. Tramway track dimensions:
a—with pylon installed between tracks; b—with lateral suspension of contact line

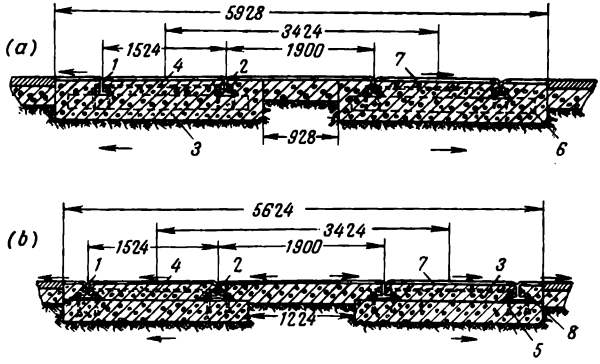


Fig. 236. Types of tramway track construction:
a—concrete sleeper foundation; b—anchored rail foundation; 1—rails; 2—bituminous mix with asbestos fibre; 3—concrete; 4—steel tie bars; 5—anchors; 6—sleepers; 7—asphalt-concrete; 8—steel plates

The minimum curve radius for a tramway track should be 20 m. On tight curves the track must be widened, the outside rail superelevated above the inside one, and the distance between the centre

lines of the tracks increased by the amount the corner of the tramcar protrudes and its middle overhangs the track.

The passage of trams creates noise and the vibration of buildings. For this reason in large cities tramcar traffic is transferred from the busiest streets to parallel ones having less intensive traffic, or it is replaced by a more modern bus or trolleybus service.

Underground communications are located under the street carriageway, sidewalks and planted strips. Streets in modern large cities are designed to provide for the accommodation and proper servicing of complex and varied underground facilities. The latter include storm water and fecal sewerage, water mains, gas pipes, heating system mains, drainage, electric high-tension and low-tension cables for various purposes, telephone, telegraph, broadcasting, fire service signalization and special-purpose cables.

The underground communications can be laid separately, i.e., with a special trench reserved for each type of facility, or grouped together in one trench, e.g., water supply, sewer, gas and central heating together; or in conduits, general or special-purpose.

General conduits are provided either for accommodating only cables or for laying cables and pipes for various purposes.

On arterial streets with a large number of underground communications good policy dictates the provision of general-purpose conduits.

All underground facilities are located at a depth sufficient for normal operation. In plan they are usually arranged parallel to the building line and to the street centre line, at various distances from the building line so as not to damage other facilities when laying new lines or repairing existing ones. When laying the communications due consideration must be given to future development, to sanitary requirements and to the convenience of taking off connections to buildings. Intersections of underground communications are arranged at different levels. Particular attention must be given to the laying of high-tension cables.

The recommended distances from underground installations to buildings, planted areas, etc., are given in Table 52.

The depth of underground communications, counting from their top, should be: for water mains up to 300 mm in diameter—0.2 m below the frost line; for larger mains—0.25 to 0.5 pipe diameter above the frost line; for sewage pipes—0.3 to 0.5 m above the frost line, but at least 0.75 m; for gas pipes—0.8 to 0.9 m; for heating system pipes—0.5 to 0.7 m; for power cables—0.7 to 1.0 m.

All underground facilities are located at least 1.5 m from the edge of the carriageway (distributing water piping—2 m) and 1 m from the outer edge of a ditch or the foot of an embankment (pipelines carrying combustible liquids—2.5 m).

TABLE 52

Name ot installations	Recommended distances from underground installations, m					
	Line of buildings	Posts and pylons—for street lighting, contact wires and communications	Tramway tracks (from the outer rail)	Overpasses, tunnels and other structures (from walls or supports)	Planted areas	
					Trees	Shrubbery
Power cables (high-tension and low-tension) and communication cables (telephone, telegraph, etc.)	0.6	0.5	2.0	0.5	2.0	0.5
Gas piping:						
low pressure	2.0	0.5	2.0	3.0	2.0	2.0
medium pressure—up to 3 kg/cm²	5.0	1.5	2.0	3.0	2.0	2.0
high pressure—from 3 to 6 kg/cm²	9.0	1.5	3.0	10.0	2.0	2.0
very high pressure—from 6 to 12 kg/cm²	15.0	2.0	5.0	15.0	2.0	2.0
Distributing water supply pipes	5.0	1.5	2.0	5.0	1.5	—
Main water supply pipes (with diameter exceeding 400 mm)	10.0	8.0	10.0	10.0	1.5	—
Storm water and sanitary sewers	3.0	3.0	1.5	3.0	1.5	—
Drainage	3.0	1.5	2.0	1.0	1.5	—
Heating system piping	5.0	1.5	2.0	2.0	2.0	1.0
Pipelines and conduits for various purposes	3.0	1.5	2.0	3.0	1.5	1.0

The recommended distances between underground facilities are given in Table 53.

The most modern method of laying underground communications is in concrete and brick *collector ducts* (Fig. 237), in which all services and communications are located. With such ducts it is not necessary to break up streets during the repair and reconstruction of the underground facilities. A certain increase in the cost of constructing a collector duct is compensated by the improvement in servicing, particularly when the underground facilities are numerous.

TABLE 53

Name of network	Minimum distance to network, m				
	Water supply line	Sewer	Gas piping	Heating pipes	Cables
Water supply piping	1.5	1.5	1.0-2.0	1.5	0.5
Sewers	1.5-3.0	0.4	1.0-2.0	1.0	0.5
Gas piping:					
low pressure	1.5	1.0	—	2.0	1.0
medium pressure, up to 3 kg/cm²	1.5	1.5	—	2.0	1.0
high pressure, from 3 to 6 kg/cm²	2.0	2.0	—	2.0	1.0
very high pressure, from 6 to 12 kg/cm²	5.0	5.0	—	4.0	2.0
Heating pipes	1.5	1.0	2.0-4.0	—	2.0
Cables:					
power	0.5	0.5	1.0-2.0	2.0	0.1-0.5
communication	0.5	1.0	1.0-2.0	2.0	0.5

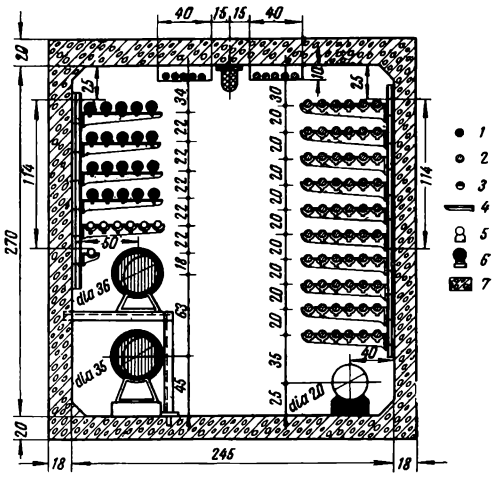


Fig. 237. Diagram of rectangular-section collector duct:

1—power cables; 2—communication cables; 3—spare; 4—metal helves; 5—water main; 6—heating mains; 7—reinforced concrete lined with damp-proof conrse

136. Street Cross-sections

Street cross-sections are worked out according to available data on the estimated traffic intensity, the character of future buildings and the location of the street in relation to the plan of the street network. When reconstructing streets, the existing street width between the building lines, the value and condition of existing buildings, and the situation of underground installations must be considered. During reconstruction the streets are usually straightened, their longitudinal gradients are eased, the carriageway and sidewalks are widened, and new vegetation is planted. Buildings

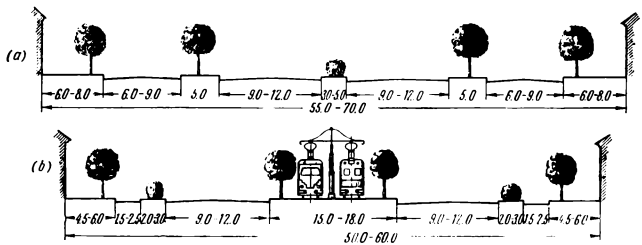


Fig. 238. Street cross-sections:
a —arterial street; *b*—major urban arterial street

of little value which hamper widening of the street are demolished, and valuable buildings in certain cases may be shifted bodily into the interior of the block. Simultaneously with reconstruction of the street the underground communications are usually rebuilt.

For each street category a variety of cross-sections may be adopted, depending on the specific traffic conditions and the width of the street.

Figure 238*a* shows a typical cross-section used for arterial streets. A feature of this particular cross-section is the separation of the part of the street for through traffic from the lanes reserved for local traffic. Sometimes the through traffic carriageway is divided by a median 3 to 5 m wide to segregate opposing traffic.

The cross-section of an arterial street provided with tramway and cycle tracks is shown in Fig. 238*b*. With such designs the cycle path may be adjacent to the carriageway or, alternatively, segregated from it by a turf-covered strip.

If there is no tramway traffic, the carriageway is separated from the sidewalk by strips covered with grass or trees. Streets in resi-

dential districts usually have a cross-section similar to the one shown in Fig. 239.

Arterial highways running through towns are gradually altered in cross-section as they near the town: the width of their carriageway

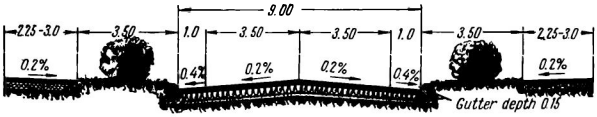


Fig. 239. Cross-section of residential block street without tramway tracks

is increased from 6-7 to 12-24 m, the structure of the carriageway is improved, the side ditches are replaced with gutters or sewers, and planted strips appear.

Fig. 240. Expressway

On the streets of large cities, where there is an appreciable traffic intensity and vehicles are frequently stopped at traffic signals, extensive delays and traffic jams may occur. For this reason, the traffic speed on streets sharply decreases and much time is necessary for the vehicles to cover the route from the town centre to the rural

highways, or for the reverse journey. To increase street capacity and improve the conditions of traffic movement, in the U.S.A., Italy, Belgium and some other countries, special expressways are constructed in cities, which are designed for high-speed traffic, up to 100-120 km/hr. These expressways are isolated from the local urban traffic; all intersections with other streets are separated. To drive onto an expressway from adjoining streets special slip roads are provided, while to simplify the construction of separated intersections and to isolate it from local traffic, the expressway is frequently located entirely in a cutting (Fig. 240). Recently, the tendency has been to build expressways at an elevated level, on trestles, since this simplifies construction, does not require the relocation of underground communications, and cuts the overall cost of construction. These considerations make expressways on viaducts or trestles more attractive. Figure 241 shows an elevated expressway passing through a suburb. A similar proposal for Chicago involves the construction of three separate three-lane trestles. Overpasses are often used for complicated interchanges in two and three levels (Fig. 242).

137. Horizontal and Vertical Layout

The *horizontal layout* of streets consists in designing the street network. When new towns are being planned the street network is selected taking into consideration the future urban traffic streams. Data on the disposition of industrial enterprises, institutions, railway stations, and public facilities make possible the estimation of the direction and volume of freight traffic. Initially the main arterial streets are located along the shortest possible routes. The other streets are planned as auxiliary ones to the main arteries in order to provide the residential quarters with convenient communication with industrial enterprises, institutions, railway stations, etc.

Usually the density of the arterial street network is 2 to 2.5 km/km^2. The distances between arteries are selected in the range of 800 to 1,000 m, and those between streets in residential quarters are 200 to 300 m. The blocks are designed with an area of 8 to 12 hectares.

The layout of streets, intersections and squares constitutes an important part of the general architectural layout of a town. The street network is designed with a view to the architectural pattern of the adjoining squares, embankments, parks, etc.

The *vertical layout* of streets consists in changing the land topography to make it correspond with engineering and architectural requirements, and also in establishing the elevations of street and square surfaces, location of underground installations, of entrances

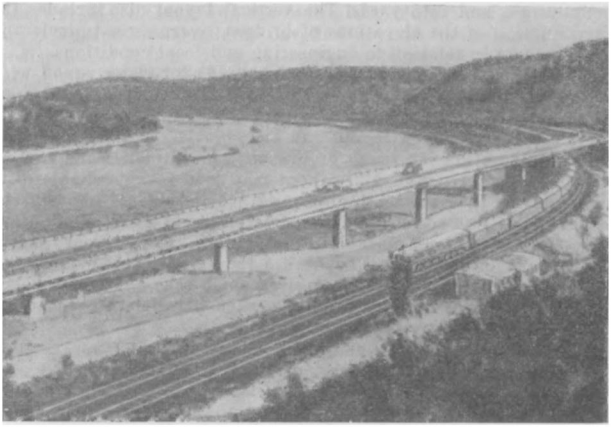


Fig. 241. Reinforced concrete trestle on single central supports

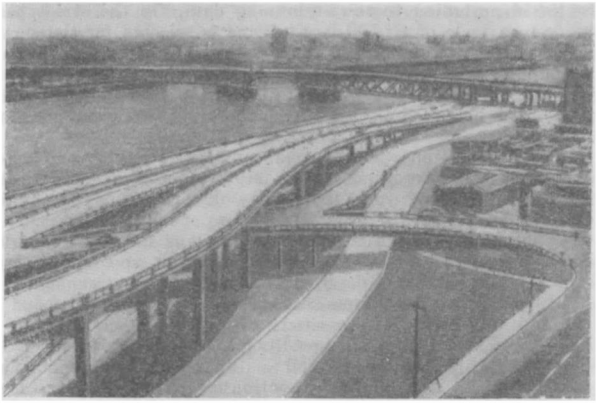


Fig. 242. Use of trestles in three-level highway intersections

to buildings, and into yards. The vertical layout also includes the determination of the elevations of bridges, overpasses, tunnels and embankments in relation to engineering and local conditions.

The vertical layout of blocks must provide for water runoff with subsequent drainage via gutters and sewers. To decrease earthworks, use must be made of the natural land topography. When natural slopes are gentle such a layout can be achieved with a rectangular block configuration. In rough ground with steep slopes and ravines the selection of a rectangular layout may lead to appreciable difficulties during the construction of buildings and underground installations. In these conditions the streets should be located along depressions and given a suitable curvature in plan, and breaks should also be introduced. This will permit a reduction in the earthworks, improve drainage conditions from adjacent blocks and create better architectural conditions for buildings sited parallel to the contour lines and rising above the street.

138. Urban Road Survey and Design in Plan and Profile

Survey of urban roads. Many aspects of surveys in urban conditions are similar to surveys for rural roads. However, there are many special features peculiar to urban conditions.

When surveying for the layout of new and the reconstruction of existing streets the general direction and plan of the street are established according to town planning data. On the street plan, usually to a scale of 1 : 500, a base line is selected for the survey work. The base line is located parallel to the street centre line and is situated so that the traffic does not interfere with the surveyors' work. The points of commencement and termination and the turning angles of the base line are set out on the ground and tied to bench marks.

In urban conditions, with a great number of underground communications and structures, great importance is attached to the precision of the survey work, the quality of which needs to be higher than for rural roads.

The route is measured along the base line, and stations are marked out at intervals of 100 m. At all characteristic spots plus points are marked. To obtain a detailed and precise street layout, at all stations and plus points cross-sections are taken and extended up to the building line. The cross-sections must be at right angles to the carriageway centre line. Levelling of the route is carried out along the base line and must be tied to all nearby bench marks. When laying out and levelling cross-sections it is necessary to determine the elevations of tramways, underground facility manhole covers, entrances to buildings and into yards, basement windows,

sewer grates, ditches, and the centre line and gutters of the carriageway. Vacant lots should be levelled beyond the building line over a distance of at least 10 m. At entrances to yards the route is levelled along the centre line of the access road or alley over a distance of 20 m from the edge of the sidewalk.

During surveys for the design of town squares, a grid is established having sides of 10 to 20 m, depending on the topography and the size of the town square. The square is then levelled by grid sections.

Simultaneously, a soil and hydrologic survey is carried out. Dug holes are excavated at average intervals of 100 m but, if required (deep cuttings, groundwater), bore wells are drilled. As a result of these investigations a soil cross-section is compiled, and the structure of the carriageway and, if necessary, drains and anti-frost heave measures are designed. The survey should establish precisely the location, size and state of existing underground installations. Use can be made of office records (plans, drawings) belonging to the relevant organization.. On the basis of the data gathered during the surveys and the initial data given in the assignment for the survey, a usual project report is compiled, while in difficult conditions first a project report and then, after some additional survey, a technical project are drawn up.

The project report should include the following items:

1. A street plan to a scale of 1 : 500 or, less frequently, 1 : 2,000, on which are indicated the overall width and dimensions of the carriageway, sidewalks, cycle paths, plantings, tramway, lamp standards, entrances to yards and a traffic movement diagram.
2. A project of the vertical layout to a scale of 1 : 500.
3. Cross-sections to the following scales: horizontal—1 : 200, vertical—1 : 100.
4. A profile to scales of: horizontal—1 : 2,000 or 1 : 1,000, vertical—1 : 200 or 1 : 100.
5. Plans of town squares, intersections and characteristic junctions to a scale of 1 : 200 to 1 : 500.
6. A water drainage project, including a plan and a profile of the sewers, manhole drawings, etc.
7. A list of the work quantities for the construction of the roadbed, carriageway, sidewalks, planted strips, drainage, etc.
8. A project of work organization.
9. An explanatory note in which the selected carriageway width, the pavement design, and the adopted methods of carrying out and organizing the work are substantiated. A special section of the explanatory note should contain data on the calculation and design of sewerage and engineering structures.
10. A financial estimate.

In the technical project the engineering solutions adopted in the project report are detailed and the quantities of work are defined more accurately, taking into consideration the specific local conditions. The work organization project includes a schedule compiled on the basis of straight-line methods of work. An estimate of the costs of construction work is included.

Design of urban roads. A street must be designed in plan and in profile with due consideration to connecting streets and squares.

The *plan of a street* is determined by its direction and the existing or planned building lines. The street design is based on engineering surveys and the surveying of the plan, profile and cross-sections. On the street plan the stations (in urban conditions stations are spaced every 100 m), cross-sections, building lines, connecting streets, entrances to buildings and yards, masts and posts, sewer grates, underground facility manholes, planted strips, tramways, the direction of underground communications and all street elements are indicated. The radii of curves are chosen as large as practicable, taking as a guide the data given in Table 54.

TABLE 54

Category of street	Radii of circular curves along centre line of street, m	
	Minimum	Recommended
Expressways	600	3,000-5,000
City arteries	400	2,000-5,000
District arteries	250	1,000-5,000
Local streets and roads:		
in residential areas	125	300-3,000
in industrial and warehouse districts	125	500-5,000
access roads	30	—

Connecting streets are joined by curves having a minimum radius of 20 m. At road intersections the curbs separating the sidewalks from the carriageway are set out along curves having a radius of 5 to 10 m, and in exceptional cases 2 to 3 m. On street corners with trolleybus traffic it is desirable to increase the curb radii to 15-25 m.

When designing the plan of the carriageway it is necessary to maintain the given width along the whole length of the street, since separate narrow sections will reduce the traffic capacity of the street over a substantial distance. On the contrary, if local condi-

tions permit, at places where it is proposed to establish public transport stops, an additional widening of the carriageway, or *turnout*, should be provided, 3.0 to 3.5 m wide. It is recommended that such stops be laid out as shown in Fig. 243.

The *profile* is usually drawn along the carriageway centre line. If tramways are laid down the middle of the street, the profile is drawn along the centre line of the space between tracks, or along

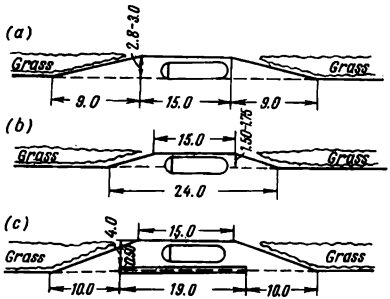


Fig. 243. Turnouts for public service vehicles: *a* and *b*—on urban streets; *c*—on streets at town exits

the top of the inner rail of a track. If the gradient of the gutter does not correspond to the gradient of the carriageway centre line, then a profile must be drawn along the gutter, which may be shown on the same diagram as the one along the centre line.

The profile shows the elevations of the stations and plus points, the grade line elevations and elevation differences, and hydrogeological and geological data.

The longitudinal gradients of streets and squares are established depending on their category (Table 55).

In mountainous and especially difficult conditions, as well as when reconstructing streets with preservation of the existing buildings, the longitudinal gradient of arterial streets may be increased by 1 per cent, and of other streets (except expressways) by 2 per cent.

The crossfall of the carriageway of streets, roads and squares is selected depending on the type of pavement (Table 56).

When curves of small radius coincide with maximum longitudinal gradients, the latter must be reduced.

To ensure the minimum sight distance, road smoothness and traffic safety, vertical curves are inserted at breaks in profile. On

TABLE 55

Category of facility	Maximum tolerable gradient, per cent
Expressways	4
City arteries	5
District arteries	5
Local streets and roads:	
in residential areas	8
in industrial and warehouse districts	7
access roads	8
sidewalks	8
Squares	3
Parking areas	2

TABLE 56

Type of pavement	Crossfall, per cent	
	Streets, through-roads, access roads	Squares, parking areas
Asphalt concrete and cement concrete	1.5-2.5	1.5
Stone block, mosaic, precast concrete and reinforced concrete	2.0-3.0	1.5-2.0
Improved light type	1.5-2.5	1.5
Intermediate types	2.0-3.0	—
Low types	2.5-4.0	—

expressways this is done when the algebraic difference of the gradients is 0.5 per cent and more, on city arteries 0.7 per cent and above, and on local streets 1.5 per cent and more.

The vertical curve radii are made as large as possible but, in any case, not less than the minimum value given in Table 57.

The carriageway cross-section may be convex or, less frequently, concave or have a straight crossfall. When the carriageway width is over 9 m a convex cross-section should be used.

The grade line location along the profile should be checked on cross-sections for each station and for each characteristic intermediate point, which permits estimating the earthwork quantities,

TABLE 57

Street category		Minimum radii of vertical curves, metres	
		Convex	Concave
Arterial	City	6,000	1,500
	District	4,000	1,000
Expressway		10,000	2,000
For local traffic in residential and warehouse districts		2,000	500
Access roads		600	200

the extent of use of the existing pavement, the provision of adequate drainage from yards and the arrangement of sidewalks at the entrances and access roads.

In street reconstruction it is desirable to use, as far as possible, the existing road pavement as the foundation for the new pavement. When easing out gradients one should avoid deep cuttings which may expose the foundations of buildings, and high embankments which require the re-arrangement of entrances and access roads and which impede water drainage from adjacent yards.

A graphic representation of the designed street surface is obtained by the method of designing the vertical layout by means of design contour lines (Fig. 244) developed by engineer V. M. Stankeyev. In this case, on the street plan drawn to the scale of 1:500 or 1:200, design contour lines are traced indicating the elevation of the carriageway, of planted strips, sidewalks and other street elements. With such a combination of horizontal and vertical design projections on the same drawing, a full description of the projected street in plan, longitudinal and transverse directions is given. Horizontal and vertical planning is carried out simultaneously. The contour line interval is usually 10, 20 or 50 cm. At particularly intricate intersections and where design gradients are small, intermediate contour lines may be drawn at 5 cm intervals.

Usually the planning of a street is commenced from the carriageway gutters in order to ensure adequate drainage of the carriageway and adjacent areas. The required gutter gradient for all surfacings is at least 0.3 per cent, and for cobblestone paving not less than 0.4 per cent. With gutter drainage the gutter elevations are established to ensure the discharge of water into adjoining or intersecting streets.

When designing underground drainage, the disposition of the drain pits and their elevation are also determined. Next the elevations

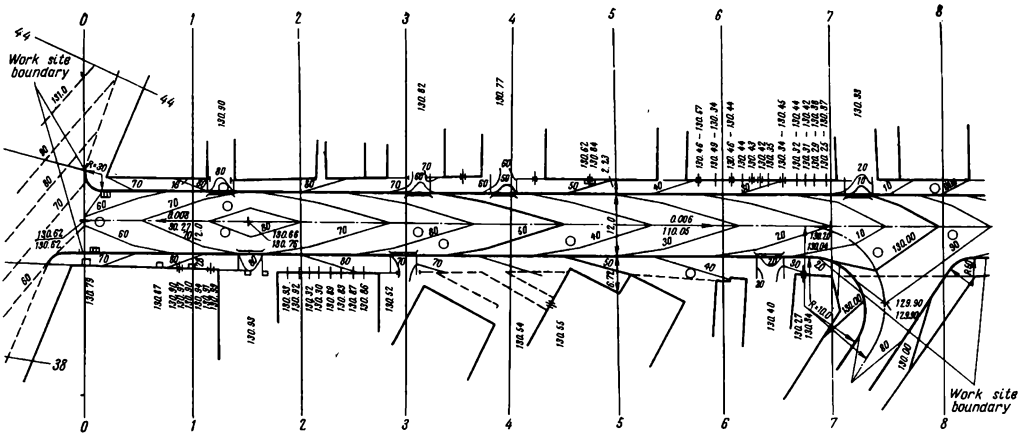


Fig. 244. Street reconstruction plan, using method of design contour lines

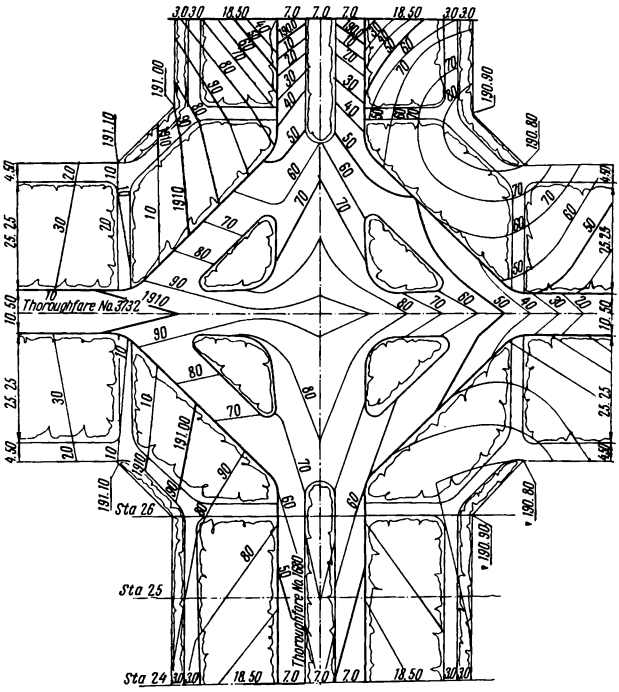


Fig. 245. Vertical planning of town square

of the centre line, the curbs, planted strips, sidewalks, etc., are determined and the design contour lines are traced.

On the vertical layout plan the elevations of the contour lines, the gradients, and also elevations at changes of gradient and of drain pits are indicated.

An example of vertical planning of a square is pictured in Fig. 245. The earthworks are calculated from the cross-sections. When designing town squares an earthwork chart is compiled, comprising a plan of the area divided into a square grid of 20 × 20 or 40 × 40 m, at

the corners of which are marked the elevation differences of the subgrade prepared for laying the pavement. The average elevation difference for each square is computed on the basis of these differences, and then the earthworks are computed.

139. Design of Street Intersections and Town Squares

Street intersections and junctions can be designed according to the various diagrams shown in Fig. 246. The type of intersection or junction is selected in accordance with the anticipated volume and character of traffic and, of course, depending on the street network plan.

At street intersections vehicular and pedestrian traffic is complicated and, therefore, it is necessary to take special measures to

Fig. 246. Types of intersections and junctions:
a—crossing at right angles; *b*—scissor crossing; *c*—T-junction; *d*—Y-junction; *e*—staggered roads; *f*—fork junction; *g*—multiway junction

ensure full safety and convenience of traffic at such places. To increase road safety it is desirable to design intersections with the necessary sight lines, as when designing rural grade intersections. However, frequently the existing buildings prevent this, and therefore at all grade intersections with heavy traffic flows traffic lights are generally installed.

Railway grade crossings are designed on a horizontal site extending in both directions for a minimum of 10 m from the extreme rail of the track.

The vertical layout of street intersections should be designed according to the importance and the category of the crossing streets and the direction of the longitudinal gradients. In this case the elevations of one street centre line may be conjugated either with the elevations of the other street centre line, or, alternatively, with those of its gutters.

On arterial streets lateral gutters cannot be laid. In certain cases a straight crossfall may be designed at intersections or junctions (Fig. 247).

For pedestrian traffic special crossings are made at road intersections, which are indicated by pavement *markers* (metal buttons, painting or strips of coloured asphalt). On streets with heavy traffic *islands* are provided for the safety of pedestrians (Fig. 248).

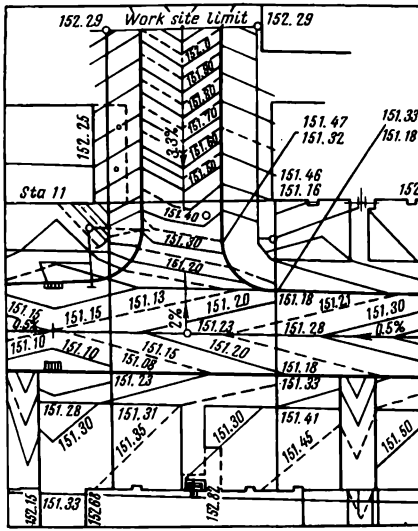


Fig. 247 Junction of a side street with a change in cross-section

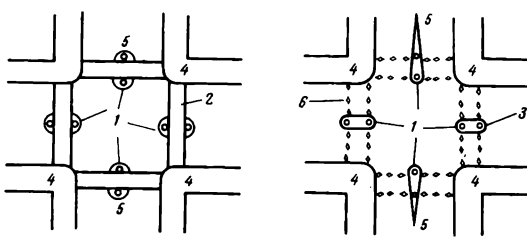


Fig. 248. Arrangement of islands at street crossings:
1—island; *2*—coloured concrete or asphalt; *3*—illuminated marker; *4*—sidewalks; *5*—carriageway; *6*—coloured studs

TABLE 58

Type of squares	Description of squares
Main town and district squares	Situated in the centre of the town, or of a district, next to administrative and public buildings. Used for holiday festivities, parades and similar events
Squares in residential districts	Situated in residential areas, next to such buildings as clubs, cinemas, libraries, shops, etc. It is recommended that gardens be provided in the residential area squares for amenity purposes, trees be planted and, when necessary, arrangements made for parking space
Squares in front of recreational buildings and halls	Situated in front of theatres, culture halls, sports grounds, exhibitions and other recreational buildings. Used for approaches to these buildings and the rapid dispersion of audiences, as well as for the passage of traffic and parking of vehicles. It is recommended that gardens be planted on such squares, which are also suitable places for the erection of monuments
Warehouse and market squares	Situated next to large buildings with department stores, arcades and markets. They serve as convenient approaches to these buildings and for accommodating parked vehicles
Station squares	Situated next to railway, road transport and river stations, as well as near to air terminals. They serve for accommodating the necessary vehicle and pedestrian traffic, provide the approaches to station buildings, and also parking space for passenger, freight and special purpose vehicles, tramcars and trolleybuses. It is recommended that these squares be covered with gardens, flower beds and other dwarf plants
Junction squares	Situated at street junctions or junctions of arterial streets and roads carrying heavy traffic. Serve for regulating and distributing traffic streams
Squares in front of bridges	Situated at the approaches of bridges and serve for ensuring a continuous traffic flow over the bridge with proper regulation and distribution of traffic at the approaches

Town squares can be classified, according to their purpose, situation, the nature of the buildings and the volume of traffic, as shown in Table 58.

The size of newly-built town squares depends on the importance and the size of the town, and on its layout and architectural pattern.

The width of the carriageway and sidewalks on squares depends on the intensity and the composition of traffic from the abutting streets and the adopted system of traffic control. A diagram of projected traffic flow and pedestrian movement is drawn on a plan of the square to a scale of 1:500. Here the number of lanes and the direction of movement of vehicles, trolleybuses, trams, buses are determined, and traffic signals, parking places and pedestrian crossings are located.

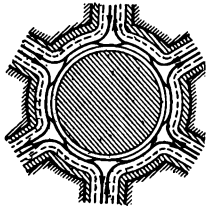


Fig. 249. Roundabout traffic flow diagram

In vertical layout of the square, depending on the general topography and the gradients of the abutting streets, a straight crossfall, a convex, concave or complicated surface form is selected that is convenient for traffic and facilitates water drainage.

Owing to the development of motor transport and the increase in the number of private cars, it has become imperative to provide parking space on squares next to stations, sports grounds, theatres, etc.

Parking sites should be isolated from the through traffic, and have a separate entry and exit. The parking capacity is estimated according to the expected number of vehicles to be parked and the space occupied by one vehicle. It is assumed that a passenger car with single-row parking occupies an area of 20 m^2, with multi-row parking—25 m^2, a bus with single-row parking occupies 32 m^2, and with multi-row parking—40 m^2.

On squares at the junction of several streets or at approaches to a bridge, parking is not planned since the purpose of the square is limited to the control of traffic streams coming from the abutting streets. The most popular regulation of traffic on such a square is by making use of a channelizing island of a circular or other regular form. The size of the island is made as large as possible, depending on the size of the square. However, the width of the thoroughfare should be sufficient to accommodate the traffic flow emerging from the abutting streets. The greater the number of abutting streets, the larger must be the thoroughfare or weaving section and the diameter of the circle. When the size of the central island is small traffic is directed around it in one way and the junction becomes a roundabout (Fig. 249).

If the shape of the square does not make it possible to design a central island, several channelizing islands are inserted, either as dividing strips or triangles, with traffic control. With heavy vehicle and pedestrian traffic, the intersections with arterial through roads are arranged at different grades by constructing underpasses or overpasses. An underpass is located in the central part of the street with space left along its sides for traffic turning right (Fig. 250).

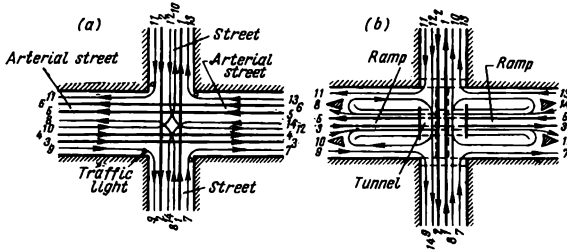


Fig. 250. Diagram of traffic flow at street intersection:
a—intersection at grade; *b*—grade separation with underpass; *1-6*—through traffic; *7, 9, 11* and *13*—right turns; *8, 10, 12* and *14*—left turns

Special tunnels can also be arranged for pedestrians when the street is wide and the traffic is heavy, as has been done, for instance, in many places in Moscow. The headroom of the pedestrian tunnel is usually 2.3 m, and its width is 5 to 8 m.

140. Drainage in Urban Conditions

With gutter drainage the water is discharged along gutters or ditches into depressions or watercourses. With a sewerage system the water accumulated in the carriageway gutters is discharged into catch basins located in the gutters, and then along sewers of the underground sewerage system into thalwegs or watercourses. A combined system is also used when part of the street is drained by means of gutters, which are subsequently discharged into a sewer.

Open ditches should not be used in urban conditions, since it is difficult to maintain them in a proper sanitary state, while the ditch slopes and bed may become scoured and cause ponding. This also involves the construction of ditch bridges or culverts. It is preferable to drain the water along gutters, which in urban conditions are reinforced with paving and by the construction of curbs. The minimum gradient of ditches and gutters is 0.5 per cent, and in exceptional cases 0.3 per cent.

An underground sewerage system is widely used in towns, especially with flat topography, when the construction of ditches and gutters is difficult. If a sewer exists the street may be planned with a slope of less than 0.5 per cent, but in this case the gutters are given a saw-tooth profile, having gradients of 0.4 to 0.5 per cent. This profile is obtained by varying the depth of the gutter within the range of 10 to 20 cm, and of the carriageway crossfall in a strip

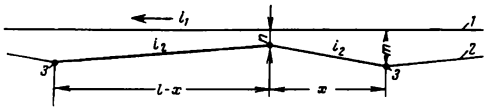


Fig. 251. Profile of gutter:
1—top of sidewalk curb; 2—bottom of gutter; 3—catch basins

adjacent to the gutter, 1 to 2 m wide. At all the depressions of the gutter saw-tooth profile (every 40 to 60 m) catch basins (drain pits) are situated.

Saw-tooth gutters are designed in the following way. In accordance with Fig. 251, showing the profile of a gutter, we have

$$\left.\begin{aligned} m &= n + \frac{i_2 - i_1}{100}(l - x) \\ &\text{and} \\ m &= n + \frac{i_2 + i_1}{100} x \end{aligned}\right\} \qquad (254)$$

where m = elevation of the curb above the grate, m
n = elevation of the curb at the watershed, m
i_1 = longitudinal gradient of the curb, per cent
i_2 = longitudinal gradient of the gutter, per cent
l = distance between catch basins, m.

From equations (254) the distances l and x are obtained

$$l = \frac{100(m-n)\,2i_2}{i_2^2 - i_1^2}\,; \qquad x = \frac{100(m-n)}{i_2 + i_1} \qquad (255)$$

When designing a drainage system in an urban locality, in the first instance the direction of the main drains is established and they are correlated with depressions and thalwegs. The drains on adjacent areas are designed to allow the runoff water to discharge into the main drain. The main sewer of an underground system is usually located along a street and parallel to the building line, but there are cases when, owing to the topographic conditions, the sewer is laid through the territory of the block.

From the catch basins in the gutters the water is discharged along lateral sewers 30 to 40 cm in diameter into the main sewer laid along the street. The water discharge from every street is directed through the network of adjacent street sewers into the outfall sewer, which discharges the sewage into a river or thalweg.

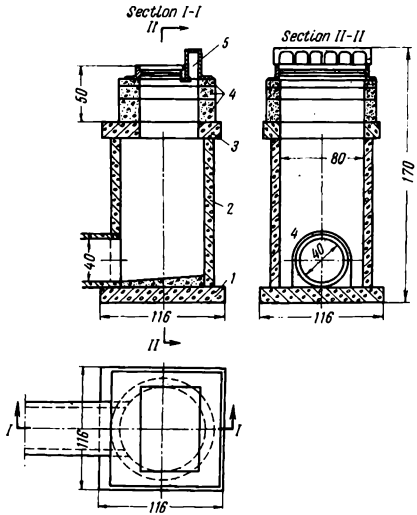


Fig. 252. Precast reinforced concrete catch basin:
1—base slab; 2—wall ring; 3—cover plate; 4—rectangular blocks; 5—metal curb with openings for water inlet

Figure 252 shows a precast reinforced concrete catch basin. The latter is of a circular section, 80 cm in diameter; the overall depth is 170 to 180 cm. The basin consists of a reinforced concrete base slab, a reinforced concrete wall ring, a cover plate and rectangular blocks. A hole is provided in the reinforced concrete ring into which is inserted a collecting pipe of 40 cm diameter. The number of rectangular blocks depends on the height of the catch basin.

The depth of the main sewer is so selected as to make possible the connection to it of all the sewers from adjacent streets. The gradients of sewers usually vary from 1 to 3 per cent. When selecting the minimum longitudinal gradient the condition must be observed

that when the drain is filled to one-third of its depth, the flow velocity will not be less than 0.75 m/sec, in order to prevent the deposition of sediment. When designing drains regard must also be given to the depth of frost penetration into the ground, so as to prevent freezing of the water. Usually the depth of sewers varies from 2.0 to 3.5 m.

The elements of the sewerage system, the distances between storm water catch basins and the diameters of sewers in urban conditions are calculated by methods of hydraulics.

With a high ground-water table in urban conditions the roadbed is drained and the table is lowered by installing drains whose design and calculation have been described above.

141. Approaches to Urban Bridges

The design of approaches to urban bridges in plan and profile differs substantially from that of approaches to rural bridges.

The situation of the bridge in plan and profile should comply with the architectural and layout requirements, i.e., should correspond to the general plan of the town and the layout of the street network adjoining the bridge. Sometimes, simultaneously with the erection of a major bridge, the adjacent streets are widened and reconstructed, and new wide motorways are built.

The bridge location is selected according to hydrogeological river conditions, the land topography, the general hydrogeological conditions and also with a view to the convenience for and the safety of traffic on the squares next to the bridge.

The alignment of the bridge crossing centre line is usually a continuation of the centre line of the bridge approaches, complying as far as possible with the requirement that the bridge should be sited at right angles to the flow of the river. It is not always possible to comply simultaneously with all of these requirements, and for this reason urban bridges are frequently built at an angle to the flow.

The horizontal location of bridges is substantially influenced by the direction of the adjoining streets. This is why in Moscow, for instance, there are several bridges built at an angle to the flow of the river (the Bolshoi Kamenny Bridge is at an angle to a normal direction of about 8°, and the Krasnokholmsky Bridge of about 35°).

The width of urban bridge carriageways is determined with a view to the prospective urban traffic flow and to the width of the adjoining streets. Since stopping on bridges is normally prohibited, the traffic makes full use of the carriageway. Therefore, the width of the bridge carriageway is made slightly narrower than that of a street.

The profile of the approaches to bridges is designed depending on the elevation of the carriageway on the bridge and on the vertical layout of the adjoining streets.

With convenient squares on the approaches to a bridge, allowing favourable traffic interchange, it is good to locate the bridge on one level with the river embankments (Fig. 253*a*). In the majority of cases navigation requirements make it necessary to elevate the bridge and to design it with a camber (hump-back bridge), the

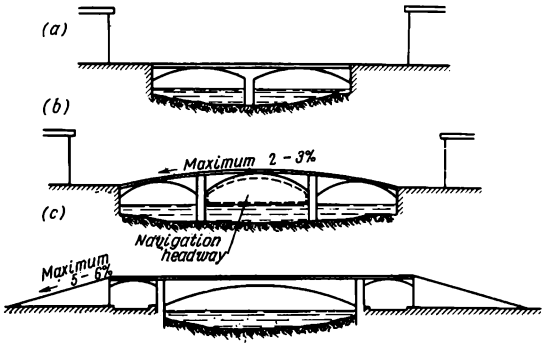


Fig. 253. Alternatives of urban bridge location:
a—on one level with embankments; *b*—hump-back bridge; *c*—crossing over embankments

maximum longitudinal gradient being 2 to 3 per cent, and vertical curves used when necessary (Fig. 253*b*). When a bridge is considerably elevated above the river embankments, it is increased in length in order to allow the embankment to pass beneath it. In this case the bridge approaches are arranged on adjacent squares or streets. The longitudinal gradients of the approaches in urban conditions do not exceed 4 to 5 per cent (Fig. 253*c*).

In urban conditions the slopes of cuttings and embankments on approaches to bridges occupy part of the street and reduce its traffic capacity, and, moreover, it is difficult to maintain them in the proper sanitary condition. Therefore the slopes are usually replaced by retaining walls.

142. Traffic Interchanges at Approaches to Bridges

The interchange of traffic at the approaches to bridges and the layout of the approach squares are designed in various ways depend-

ing on the situation of the approaches to the bridge and the traffic intensity on the adjacent streets.

When a bridge is sited at the same level as the embankments, the square in front of the bridge is located next to it. With small volumes of traffic along the embankments, the interchange may take the form of a simple intersection with traffic control (Fig. 254*a*).

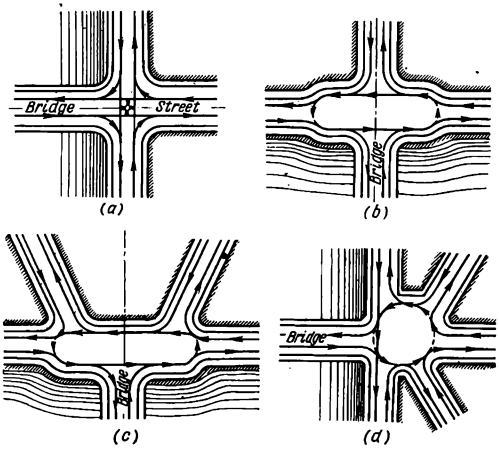
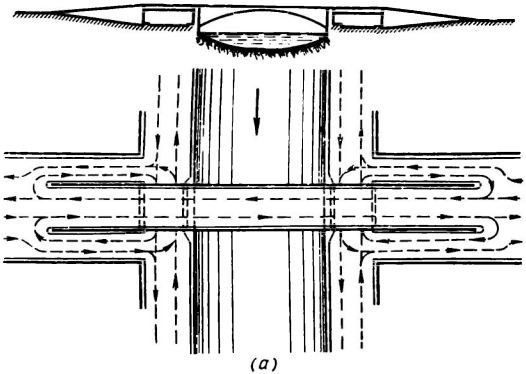


Fig. 254. Traffic interchange at approaches to bridges:
a—as on normal intersection; *b* and *c*—with central island; *d*—roundabout

If the traffic on the embankments is heavy, then the latter are widened at the approaches to the bridge. In front of the bridge vehicles are diverted around an elliptic island, which improves traffic conditions along the embankment (Fig. 254*b* and *c*). If the traffic on several streets approaching the bridge is heavy, the vehicles are directed one way around the square, which forms a large roundabout (Fig. 254*d*).

The most perfect form of traffic interchange is when the bridge approach roads pass over the river embankments. The vehicles moving along the embankment pass beneath the approach viaduct. Because of the substantial height of the bridge approaches these must be located along the adjoining streets, while the approach square is located beyond the limits of the approaches. The vehicles drive



(a)

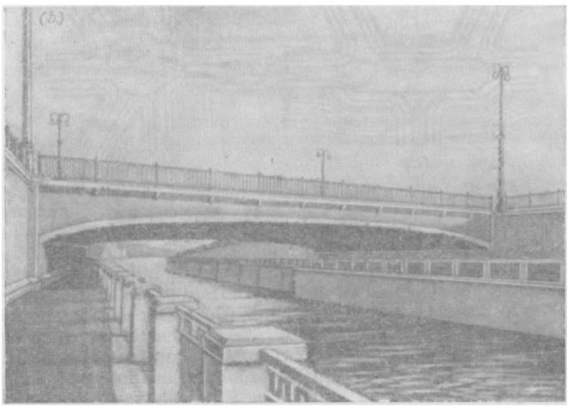


Fig. 255. Traffic interchange when a bridge crosses over the embankments (*a*), and location of sidewalks under a bridge (*b*)

over from the bridge onto the embankment along the approach road retaining walls (Fig. 255*a*). The gradient of these roads should not exceed 4 to 5 per cent.

143. River Embankment Layout

When planning embankments it is necessary to design a thoroughfare along the river bank and to reinforce the banks. The decoration of the embankment should harmonize with the architectural ensemble along the river.

For the arrangement of the embankments in plan, designers must be guided by the *control line*—the line of intersection of normal

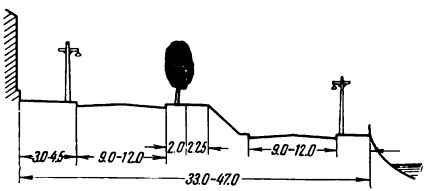


Fig. 256. Cross-section of two-level embankment

water level with the bank slopes. The control line is selected with the aim of giving the embankment a flowing contour, and of making the river banks parallel, if possible. In addition, the control line is coordinated with the building line on the embankments, in order to lay out streets of the required width (Fig. 256).

To fulfill the above requirements it is necessary to cut or fill the banks as required and to level out the adjoining territory. The river embankment should be located sufficiently high to prevent the flooding of adjacent areas, and protect the buildings and road structures from the damaging effects of ground water. The river banks are fastened against erosion by planting shrubbery and trees, by turfing and, when the velocity of the water is high, by a single layer of paving over a layer of sand or, preferably, rubble. Better fastening of the slopes ensuring stability and the proper shape of the banks is achieved by the use of large stones, concrete slabs, masonry or asphalt concrete.

In large cities the banks are protected with stone, concrete, and reinforced concrete retaining walls. In some cases concrete and reinforced concrete revetment walls are used.

In Fig. 257 a reinforced concrete embankment *revetment wall* is shown, which has been erected where the bank does not need to

be elevated and where the bank slope is sufficiently stable. The reinforced concrete slab on the top is covered by a granite coping. The wall, the lower part of which has a slope of 45 degrees, rests on a rock base. For the discharge of ground water a drain is laid behind the wall, with outlets located at intervals along the wall.

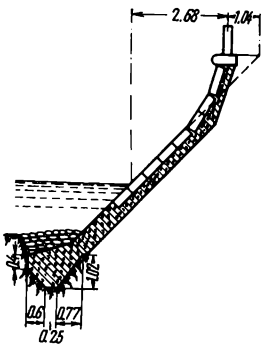


Fig. 257. Design of embankment revetment wall

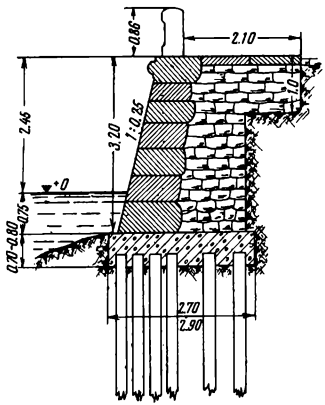


Fig. 258. Solid retaining wall on pile foundation

Above the wall a stone parapet is erected 0.9 m high or, alternatively, a metal railing. In certain cases, if it is necessary to fill the bank, solid retaining walls may be constructed on pile foundations (Fig. 258).

For the passage of all types of cables special ducts are provided in the embankments. Sewers are designed to discharge above the river water level in order to avoid backing-up of the sewage network. Sometimes the water outlet is situated below the water level in order to improve the appearance of the embankment and to prevent freezing of the outlet in winter.

Owing to the gentle falls of rivers, which are followed by the embankments, the longitudinal gradient of the embankment is very small. For this reason the carriageway gutters are given a saw-tooth profile with a minimum gradient of 0.5 per cent. The gutter grates are located approximately every 50 to 60 m, discharging the water into the river.

Where the embankment is provided with a wide carriageway, the cross-section is cambered, and when the carriageway width is 10 m or less, a straight crossfall is adopted with a gradient of 1.5 to 2.5 per cent towards the river. Inside the embankment parapet or railing a sidewalk up to 5 m wide is constructed.

Where there is urban river passenger traffic special landing stages are provided along the embankment. Aesthetically pleasing ramps and grand-stands are also arranged on stretches intended for water sport competitions.

If the bridge crossing the river is somewhat higher than the embankment, then the embankment wall and sidewalks are gradually raised to its level. With a substantial bridge elevation, the embankment sidewalk may pass beneath the bridge (see Fig. 255*b*).

If the embankment traffic is to pass beneath the bank spans of the bridge, then the clearance under the bridge should be 4.5 to 5.0 m. In this case the embankment wall will adjoin the bridge abutment.

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